SERGENT, HAUSKINS & BECKWITH

GEOTECHNICAL INVESTIGATION REPORT
CONVEYOR LINE CORRIDOR
KENNECOTT UCD MODERNIZATION PROJECT
SALT LAKE COUNTY, UTAH

Prepared For:

Kennecott
c/o Ralph M. Parsons Company
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February 3, 1986

SHB Job No. E84-2011J

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Attention: Mr. Peter W. Harvey
Project Manager

Re: Geotechnical Investigation Report
Conveyor Line Corridor
Kennecott UCD Modernization Project
Salt Lake County, Utah

Gentlemen,

Our Geotechnical Investigation Report for the site of the referenced project is herewith submitted. The report presents results of the field exploration program and the results of laboratory analysis, along with our evaluations and recommendations for foundation design and other earthwork elements of the project.

Should any questions arise concerning this report, we would be pleased to discuss them with you.

Respectfully submitted,

Sergent, Hauskins & Beckwith Engineers

By Paul Kaplan
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1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm at the site of the proposed ore conveyor system for the Kennecott Utah Copper Division (UCD) Modernization Project. The ore conveyor system traverses a total length of approximately 4.9 miles from the Bingham Canyon open pit mine to a stockpile at the proposed concentrator site located approximately 1.25 miles north of Copperton, Utah. The portion of the proposed ore conveyor system covered by this report extends for a length of approximately 1.8 miles from the 5490 railroad tunnel just south of Bingham Canyon to the proposed concentrator site.

The objective of this investigation was to evaluate the physical properties of the subsurface soils and bedrock underlying a portion of the proposed ore conveyor alignment in order to provide recommendations for foundation design, slab support, design of subgrade walls, access road surfacing, site grading and other earthwork elements of the project.

The investigation and report preparation were conducted under the supervision of George H. Beckwith, P.E., Project Manager and Darrel L. Buffington, P.E., Project Engineer of this firm.

2. Project Description

Details of the proposed construction were provided by Bernard Pollack of the Ralph M. Parsons Company and Mr. Peter W. Harvey of Kennecott.

It is understood the project involves an ore conveyor system capable of transporting up to 10,000 tons of ore per hour. The conveyor belt will be 72 inches in width with supports spaced on 20 foot centers. Dead plus live loads are anticipated to be on the order of 700 pounds per lineal foot.

The conveyor system originates in the Bingham Canyon open pit mine and runs through the existing 5490 railroad tunnel for a distance of approximately 16,400 feet. The portion of the conveyor system adressed this report extends from the exit portal of the tunnel and runs overland for a distance of approximately 1.8 miles to the coarse ore stockpile at the proposed concentrator site which is located approximately 1.25 miles north of the town of Copperton, Utah.

Three portions of the conveyor system will be elevated supported on trusses at the following locations (Refer to Sheets 1 & 2):

Conveyor No. 6 from station 163+82.35 to station 173+52 where the alignment crosses the railroad yard at Bingham Canyon. Vertical and horizontal components of the structural loads at the supports will be 220 kips and 55 kips, respectively.

Conveyor No. 7 where the alignment crosses Utah State Highway 48 near the town of Copperton.

The final portion of Conveyor No. 8 as it rises approximately 170 feet above grade to the coarse ore storage area.

Two transfer stations are planned along the proposed alignment at changes in conveyor direction. The transfer stations are relatively light structures. Foundation loads on the order of 3000 pounds per square foot are anticipated.

Extensive site grading will be required along the overland portion of the conveyor alignment. Cuts on the order of 75 feet in depth and fills on the order of 60 feet are anticipated. Retaining structures will be required at several locations.

3. INVESTIGATION

3.1 Review of Literature & Previous Mapping

Prior to and in conjuction with the field exploration program, the following data provided by Kennecott Utah Copper Division, were reviewed:

A. Utah Copper Division Modernization maps of the mine and concentrator site areas at various scales.

B. Aerial photographs of the concentrator site area.

C. Well logs from the Copperton culinary water wells.

D. Historical rainfall and temperature data at the Bingham Canyon weather stations, 1941 to 1982.

E. Geologic mapping and literature published by government agencies and professional societies.

Jaren Swenson, Kennecott Utah Copper Division geologist, provided background information and unpublished results of Kennecott geologic mapping of the concentrator site areas during an informal discussion with Ralph E. Weeks, P.G., senior geologist and Paul V. Smith, P.G., of SHB at the Bingham Canyon Geology Office on May 17, 1984.

3.2 Aerial Low-Sun-Angle Reconnaissance

Aerial low-sun-angle reconnaissance and photography of the concentrator site area, the Lake Bonneville shoreline east of the site area, and the northern terminus and western flank of the Oquirrh Mountains were carried out on May 16, 1984. Participants in the aerial reconnaissance flight were Ralph E. Weeks, P.G., senior geologist, and Paul V. Smith, P.G., and Robert A. Whitney, staff geologist of SHB.

1:24,000 scale color infrared aerial photography of the site region, provided by the Utah State Office of the Bureau of Land Management, was stereoscopically reviewed by Robert A. Whitney, staff geologist. This analysis did not reveal the presence of apparent Holocene fault scarps or other potential geologic hazards which have not been previously noted in the literature or the aerial low-sun-angle reconnaissance.

3.3 Subsurface Investigation

The subsurface exploration program consisted of exploratory borings and surface seismic refraction surveys.

A site plan showing the location of the borings and seismic refraction profiles is presented on Sheet 1 in the map pocket at the end of this report.

All soils were classified by the Unified Classification System (ASTM D2487), which is summarized in Appendix A. Terminology and coding used in the description of rock is also presented in Appendix A, along with the boring logs and a short description of drilling methods employed.

All borings were backfilled with cuttings subsequent to drilling.

Supervision of the test drilling was performed by Bryan J. Bowser, staff geologist of this firm.

3.3.1 Exploratory Drilling

A total of nine exploratory borings were advanced to depths ranging from 20.2 to 56.5 feet below the existing ground surface utilizing 6-5/8 inch diameter hollow stem auger. Standard penetration testing and thin-walled Shelby tube sampling was performed at intervals of 5 feet or less in the borings.

3.3.2 Seismic Refraction Profiling

Seismic refraction surveys were performed by Michael L. Rucker and Bryan J. Bowser, staff engineer and staff geologist of this firm on August 14 through 16, 1985. Seismic refraction surveys were performed at selected locations where deep cuts, fills, or other earthwork operations are proposed. A total of 12 compression and two shear wave refraction lines were performed. Locations of the surveys are shown on the site plan included as Sheet 1 in the map pocket.

Refraction seismic exploration was accomplished using the 12-channel EGG Nimbus signal enhancement seismograph in conjunction with cabling to provide 25-foot spacing between the 12 geophones for a total coverage of 300 feet per setup. This configuration gives a depth of investigation of as great as 100 feet.

A sledgehammer with a metal plate target served as the seismic energy source. For each refraction line setup, a "forward" and "reverse" velocity profile was obtained to assist in subsurface coverage and interpretation of the data. The hammer as then moved to the middle of the geophone array and a profile to collect near surface data in the middle of the set up was obtained. Shear waves were generated by excavating a small pit, placing the metal plate target on each side of the pit parallel to the axis of the geophone array, and striking the plate in a direction perpendicular to the geophone array axis using a horizontal stroke of the hammer. The supplied mapping and field stakes were used as location control for the refraction survey setups. Refraction line locations are shown on Sheet 1, while time travel plots and interpretation of the results for the twelve refraction lines are presented in Appendix C.

3.4 Laboratory Testing

To aid in the classification of soil and rock materials; grain-size distribution, moisture content, and Atterberg Limits determinations were performed on selected standard penetration test samples.

Chemical analysis was also performed on selected soil samples by the University of Utah Research Institute, Earth Science Laboratory to evaluate corrosion characteristics of the soils. The analysis consisted of determinations for pH, conductivity and percent sulfate.

Results of moisture content determinations are shown on the boring logs presented in Appendix A. The results of all other laboratory tests are presented in Appendix B.

4 GEOLOGY

4.1 Regional Setting

The regional geologic setting was discussed in detail in our Final Geotechnical Investigation Report for the Concentrator and Allied Facilities (SHB Job No. E84-2011) as well as the Preliminary Geotechnical Investigation Report for the Tailings Pipeline Corridor (SHB Job No. E84-2011B) for the Kennecott UCD Modernization Project dated December 12, 1985 and September 13, 1984 respectively, and will only briefly be discussed herein.

The proposed ore conveyor system covered by this report extends for a length of approximately 1.8 miles from the exit portal of the existing 5490 railroad tunnel near Bingham Canyon to the proposed concentrator site. The elevation ranges from 5500 feet, slightly south of Bingham Canyon, to 5830 feet at the concentrator site.

The conveyor line corridor is located on the eastern flank of the northern portion of the Oquirrh Mountains. The Oquirrh Range is a typical north-south trending block faulted range in the eastern part of the Basin and Range physiographic and seismotectonic province. Bedrock beneath the site is composed of Late Paleozoic and Tertiary-aged sedimentary and volcanic units, unconformably overlain by the Harker's Alluvium of Early Pleistocene age (Tooker and Roberts, 1971; Swensen, 1975; Davis, 1983). The Oquirrh Mountains probably have been thrust eastward (or underthrust from the east) along a deep-seated plane that is now offset by normal faults. The thrust plane probably also exists at depth under the Jordan Valley as well (Davis, 1983).

The Wasatch Front, which marks the boundary between the Basin and Range and Colorado Plateau provinces, lies about 15 miles to the east of the Oquirrh Range, separated from it by the valley of the Jordan River, which drains northward to the Great Salt Lake. The valley floor is covered by shoreline facies and lake-bottom sediments of Mid to Late Pleistocene Lake Bonneville and Late Quaternary to Holocene alluvial deposits of the Jordan River.

Relief on the western and northern flanks of the Oquirrh Range is considerably more pronounced than on the eastern flanks. The western frontal fault system of the Oquirrh Range is composed of steeply west-dipping normal faults, and a number of Holocene-aged fault scarps have been identified(Bucknam, 1977; Anderson & Miller, 1979, 1980). On the eastern flanks of the range, however, extensive alluvial fans of early Pleistocene age have formed which reveal no evidence of fault activity at the surface. No Holocene-aged fault scarps have been observed along the eastern flanks of the Oquirrh Range or in the adjoining Jordan River Valley(Bucknam, 1977; Anderson & Miller, 1979, 1980). Normal faults cut Precambrian to Middle Paleozoic-aged bedrock beneath the Jordan Valley (Hintze, 1980), but they have no surface expression.

4.2 Site Geology & Geotechnical Profile

The oldest bedrock units exposed in the area of the site are Permian-aged sedimentary rocks of the Bingham Sequence which are extensively exposed in the northern Oquirrh Mountains, 0.7 to 1.5 miles west of the conveyor line corridor. They are not exposed, nor were they encountered in any of the exploratory borings along the conveyor line corridor. These rocks are divided into three formations, from oldest to youngest: the Curry Peak Formation, the Freeman Peak Formation and the Kirkman-Diamond Creek Formation (Sheet 1). The Curry Peak Formation is principally composed of light gray to light tan calcareous sandstone, siltstone, quartzite and orthoquartzite, with interbedded limestone and chert pebble conglomerate and breccia.

The Freeman Peak Formation overlies the Curry Peak Formation and consists primarily of light gray to brownish tan calcareous quartzite and orthquartzite with minor interbeds of calcareous sandstone and platy argillaceous siltstone and shale.

The Kirkman-Diamond Peak Formation overlies the Freeman Peak Formation and consists of thinly laminated limestone and arenaceous limestone, overlain by light gray to tan or white calcareous sandstone, with minor interbedded dolomite and dolomitic limestone.

Two thrust faults with west-over-east displacement cut rock of the Curry Peak and Freeman Peak Formations in Bingham Canyon about 1 mile west of the conveyor line corridor. In addition, the rocks are folded and overturned in places. These units and other rocks of Precambrian to Late Paleozoic age extend eastward at depth from the Oquirrh Mountains beneath the Jordan Valley, where they are folded about north-trending axes and cut by north-trending faults (Hintze, 1980).

The Paleozoic rock of the Oquirrh Range is cut by intrusions of Tertiary age in Bingham Canyon and unconformably overlain by Tertiary sedimentary and volcanic rock along the east flank of the range. The Tertiary rocks are in turn overlain by alluvium to the east. Tertiary volcanic rock is well-exposed in outcrops along the conveyor line corridor and was encountered in Borings C-1 and C-7 at depths of 40 feet and 13 feet, respectively, and possibly in Boring C-8 at a depth of 50 feet beneath the Harker's Alluvium (Sheet 1). Dikes of undifferentiated intrusives of Tertiary age cut Tertiary volcanics and Paleozoic rock at several places, and alluvial and lacustrine tuff, marlstone, conglomerate and limestone of Tertiary age outcrop on the lower slopes of the Oquirrh Range to the north and east of the conveyor line corridor.

Four volcanic rock units of Tertiary age are differentiated in the site area (Swensen, 1975). The exact sequence of emplacement of these units is uncertain.

Latite flows, the most abundant flow rock exposed in the volcanic sequence, outcrop in two areas northwest and southwest of the conveyor line corridor. The latite (Tvlp) is composed of biotite-hornblende latite with less common augite-biotite latite. Flow structure is generally evident. Hornblende-latite (Tvhl) flows are exposed in one area on the north slope of Bingham Canyon near the south end of the conveyor line corridor. This rock is a hornblende-biotite latite with more abundant hornblende than the latite (Tvlp).

Extensive exposures of latitic breccias, which may reach thicknesses of several thousand feet, occur along the eastern flank of the Oquirrh Range, generally dipping eastward at 10 to 15 degrees (Swensen, 1975). Two separate units are distinguished in the site area, herein referred to as latite breccia (Tvlb) and laharic breccia (Tvb). The latite breccia is exposed in Bingham Canyon near the south end of the conveyor line corridor and consists of subangular to subrounded fragments, up to 2 inches in length, of biotite-hornblende latite, augite-biotite latite, quartzite and limestone, set in a dense, mostly aphanitic latite matrix. The fragments make up about 50 percent of the rock.

The rock unit herein referred to as laharic breccia is by far the most extensively exposed Tertiary unit along the conveyor line corridor. It consists of water-deposited tuff, sandstone and gravel, and commonly contains lithic fragments up to 12 feet or more in size. The most common fragments are biotite-hornblende latite and augite-biotite latite, andesite, basalt and quartzite, set in a matrix of crystal-lithic tuff. Water-lain beds, up to 30 feet thick, of latitic crystal-lithic tuff, tuffaceous sandstone and conglomerate are interbedded with the breccias.

Laharic breccia (Tvb) bedrock encountered in Borings C-1 and C-7 consists of angular to subrounded rock fragments, commonly less than 1 foot in size set in a matrix of crystal-lithic tuff. The rock, which is white to grayish green and olive in color, is moderately to highly weathered, especially the upper 8 or 9 feet as noted in Boring C-7. Both the lithic fragments and finer grained matrix are soft to moderately soft. The rock weathers to form a low to medium plasticity sandy clayey silt. Bedrock is exposed along the conveyor corridor from about grid line N18300 to grid line N22600, but is usually veneered by alluvium and weathered bedrock as noted in the log of Boring C-7 and the seismic surveys performed in this area.

An erosional unconformity separating the laharic breccia unit from the overlying Harker's Alluvium dips down underneath the site in Barney's Wash with the unit being encountered in Boring C-7 at about 13 feet. The rock at Boring C-7 is highly weathered to moderately weathered from 13 to 21.5 feet.

A series of generally northeast-trending latitic dikes (Tiu) cut both the Tertiary volcanic rocks and the Paleozoic Bingham Sequence. Two small dikes are exposed near the south end and middle of the conveyor line corridor. A normal fault with a northwest trent cuts one of the dikes 1000 feet west of the conveyor line corridor.

The most significant unit, next to the laharic breccia unit, in terms of site geology is the Harker's Alluvium of probable Early Pleistocene age (Sheet 1). It occurs as extensive, fan deposits on both sides of the Oquirrh Mountains. The thickness is unknown, but probably reaches at least 200 to 250 feet or more in places. It unconformably overlies the Tertiary volcanic rocks along the east flank of the Oquirrh Range, including the site area. The Harker's Alluvium is highly variable, but, in general consists of unconsolidated, poorly sorted layers containing boulders up to several feet or more in diameter alternating with clayey sands and gravels; clean, well-sorted sands and gravel, with well-developed cross bedding; and occasional layers of silty or sandy clay and clay. The rapidly changing and cross-cutting nature of the various deposits within the Harker's Alluvium results in a highly variable geotechnical conditions along the conveyor line corridor.

The character of the alluvial materials encountered in the exploratory borings demonstrates the variable subsurface conditions. Silty clays, sandy clays, and gravelly clays with medium plasticity are the predominant material present and were encountered in all of the borings. A layer of silty to sandy clay with numerous gravel and cobble-sized rock fragments occurs to a depth ranging from 2 to 7.5 feet over nearly the entire site. This is underlain by additional alluvial materials consisting of clayey and silty gravels with interbeds of silty sand, clayey sand, and sandy clay ranging in thickness from 2 to 8 feet.

Most of the alluvial materials encountered during the exploratory drilling program were in a slightly moist to moist condition, except for those in Bingham Canyon which were moist to very moist. A weak to moderate lime cementation was encountered in most samples with stronger cementation in the silty and clayey materials.

Below the upper few feet, the granular soils are generally dense to very dense, while the finer grained deposits are generally very firm to hard. However, occasional softer layers of clayey silts, silty sands and clayey sands are present at depth.

Man made fill was encountered along the conveyor line corridor in Bingham Canyon. However, its exact areal extent is not known, and it is not differentiated from the alluvium on the geologic map of the site (Sheet 1). Fill was encountered in Borings C-1 through C-4 ranging in thickness from 4.5 to 40 feet and consists of a variable mixture of silty and clayey sands, sandy clays, and silty and clayey gravels with cobbles and boulders. The fill was generally moist to very moist and moderately firm to hard, with layers of nonplastic to highly plastic material. Trash commonly occurs in the fill and consists of slag, railroad ties, wire, and concrete pipe.

Color of the fill varies from dark brown and black to reddish brown and olive green.

4.3 Interpretation of Seismic Refraction Data

Due to the general and approximate nature of the geophysical techniques applied here, all depths, locations and refraction velocities presented must be considered approximate. However, trends and general conditions are apparent.

Geologic interpretation of the refraction survey data is presented in Appendix C and Sheet 2. In general, the surface material layer is about 5 to 15 feet thick. In a few areas, this layer is about 20 feet thick, and in other areas with rock outcrops at the surface, it is not present. Line CS-1 shows this material to extend to a depth of 40 or more feet. Compression wave velocities are in the range of 1,400 to 2,000 feet per second.

Below the low velocity surface layer, there is a range of increasing material velocities found throughout the site as progressively less weathered material is encountered at depth. The top of this higher velocity layer ranges from occuring immediately below the surface layer to being exposed at the surface at some refraction line locations. It is present to depths of from 30 feet to beyond the depth of investigation. Material compression wave velocities generally range from about 2,300 feet per second to in excess of 5,000 feet per second. These velocities probably reflect the reduction in the degree of weathering in the material with depth.

Velocities in excess of 7,000 feet per second were identified at several locations and depths. Of greatest interest are several zones where the compression wave velocities indicate a potential need for blasting if excavation through those zones is required. Such zones were identified at lines CS-6, CS-7, CS-8, and CS-12. Material compression wave velocities in these zones may be in the range of 8,000 to 10,000 feet per second. Variable topography and inconsistent surface material layers both complicated and reduced the confidence in the interpretations of these lines.

Shear wave velocities were obtained at lines CS-13 and CS-14, which correspond to lines CS-11 and CS-5, respectively at which locations compression velocities were evaluated. These lines were in streambed areas where considerable fill is anticipated. Results were generally poor, but some interpretable data was collected. Surface layer shear wave velocities to depths of about 10 feet to 15 feet ranged from about 700 to 1,000 feet per second. At depths of about 10 feet to 40 or 50 feet, shear wave velocites ranged from about 2,200 to 2,800 feet per second. One reading, at a depth of 40 feet, indicated a shear wave velocity of about 5,000 feet per second.

Only one borehole was advanced along a refraction line for this investigation. Boring C-7 was located at the junction of lines CS-7 and CS-8, and refused at a depth of 23 feet in material with a compression wave velocity of about 3,500 to 4,500 feet per second. Results of this refraction survey are similar to the results from an earlier refraction seismic survey performed by this firm in areas north of the proposed concentrator site (SHB Job No. E84-2011, dated December 12, 1985).

4.4 Groundwater

Small zones of perched groundwater were encountered in Borings C-1, C-2 and C-3 at depths of 27 feet, 50 feet, and 9.7 feet, respectively. Although many of the clayey zones were quite moist, no free groundwater was encountered in any of the other borings.

According to Hely and others (1971), the general groundwater table beneath the easterly portion of the site is at about elevation 5200 feet or about 350 feet to 400 feet below the ground surface.

Chemical analysis performed on selected soil samples in Borings C-1 and C-2 within the fill portions of the borings indicated possible contamination by leachates. Borings C-1 and C-2 had pH values of 4.80 and 5.32, conductivity values of 6250 and 3700 micromhos/cm, and 0.50 and 0.54 percent sulfate, respectively. Boring C-3 which had a fill depth of 7.5 feet, had a pH of 4.28, a conductivity of 4950 micromhos/cm and a percent sulfate of 0.18 at a depth of 15 feet, or 7.5 feet below the fill. Borings C-6 and C-8, which were not located in fill, had higher pH values, lower conductivities, and smaller percentages of sulfate than those in fill areas. Borings C-6 and C-8 had pH values of 8.62 and 9.50, conductivities of 1025 and 810 micromhos/cm and 0.033 and 0.005 percent sulphate, respectively (Appendix B). The fill areas with low pH and high conductivity and sulfate content appear to be more conducive to corrosion of steel and concrete than the areas overlying alluvium and bedrock.

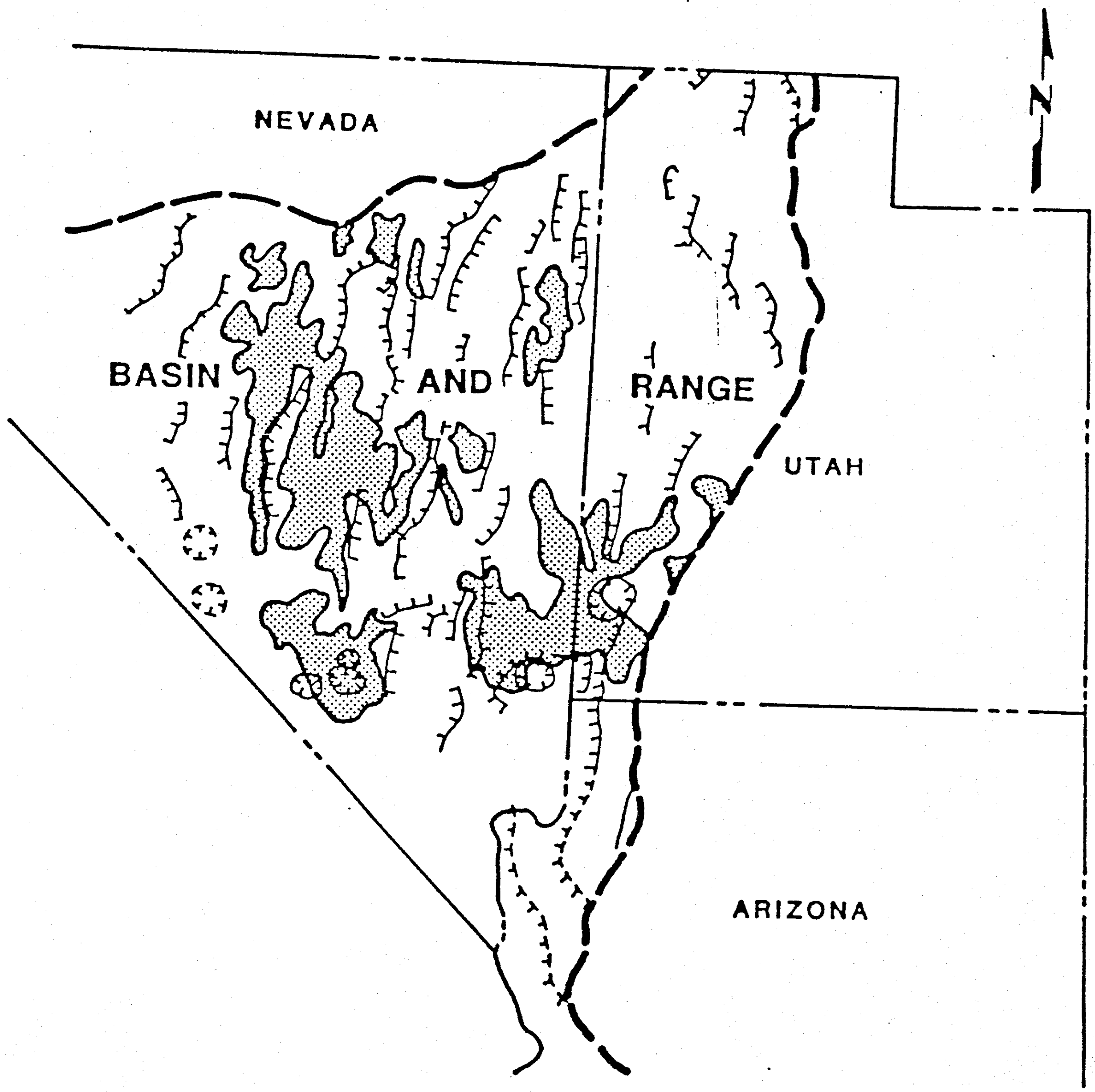
5. SEISMIC HAZARD EVALUATION

5.1 Seismic Evaluation

This seismic hazard evaluation is based on review of the literature for fault activity and structure of the site region, and low-sun-angle aerial and ground reconnaissance of the site area and regions to the east and west of the site, excluding the well studied Wasatch Front. Active faults which might affect earthquake design parameters at the site are delineated and characterized from these studies.

5.2 Regional Seismotectonic Setting

The site is located near the eastern boundary of the seismically active northern portion of the Basin and Range Province of the western United States (Figure 1). The Basin and Range Province is an extensional system of horsts and grabens forming over an extending crust about 20 to 25 kilometers thick in the site region (Smith, 1978). Systems of high-angle, normal faults separate the horsts and grabens, and similar fault systems with lesser displacements are present within the horst and graben blocks. Zoback and Zoback (1980) indicate extensional direction of the site region is in question, with various methods of determining extensional directions resulting in trends ranging from about N80W-S80E to S70W-N70E. Focal mechanism solutions for small to moderate earthquakes in the site region indicate an extensional direction of about S80W-N80E (Arabasz and others, 1979). The seismotectonics of the site region



NOTE: FAULTS SHOWN DATE FROM LATE CENOZOIC AND QUATERNARY.

LEGEND

TERTIARY AND QUARTERNARY VOLCANIC ROCKS

PHYSIOGRAPHIC BOUNDARY

GENERALIZED LATE MESOZOIC-CENOZOIC TECTONIC MAP OF BASIN AND RANGE
(after Arabasz and others, 1979)

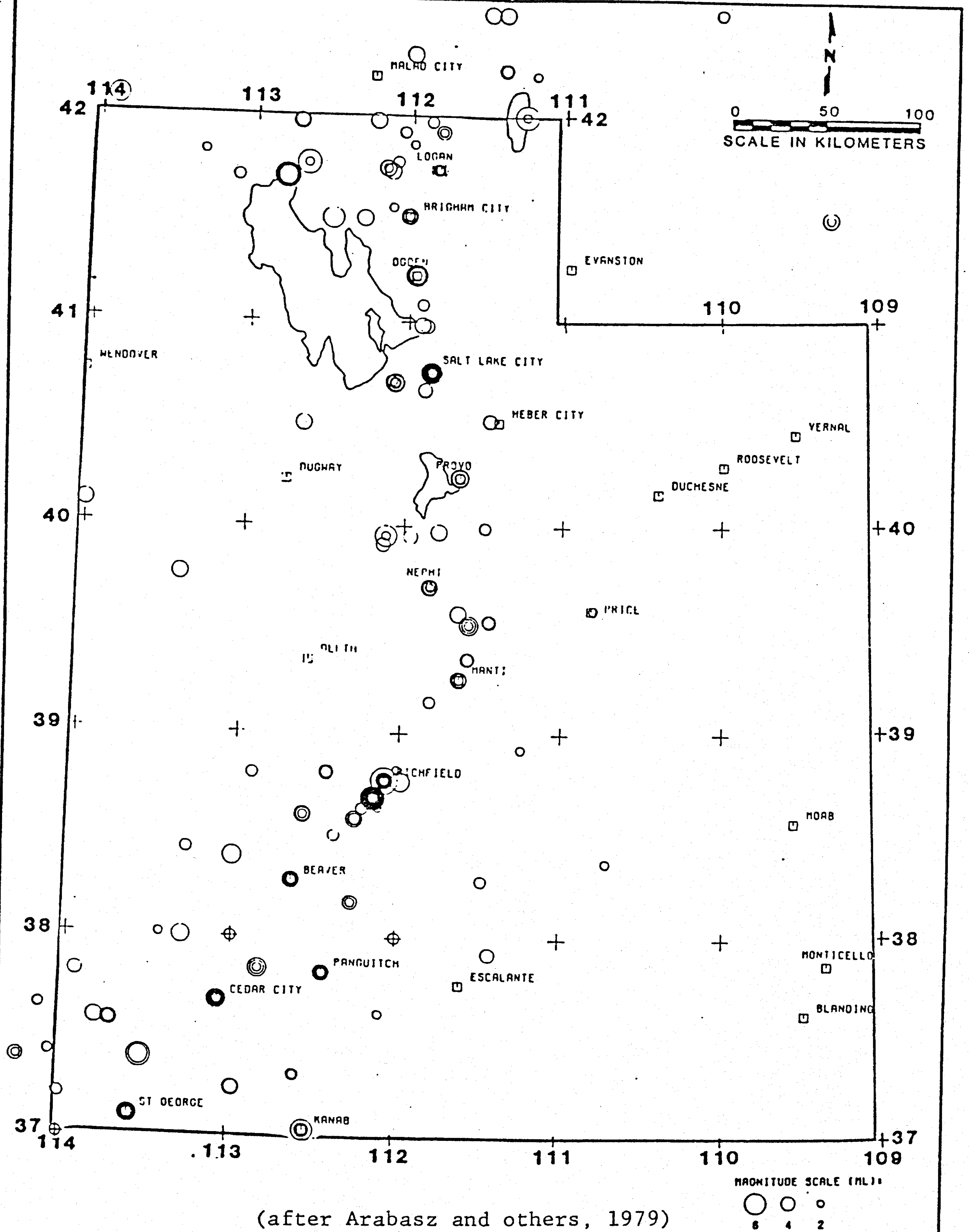
1980. An additional computer search for seismicity within the state of Utah was accomplished for this study. Figures 2 and 3 summarize this research. These data detect earthquakes of about magnitude 4.5 and larger from 1850 to June, 1962; larger than about magnitude 2.5 from June, 1962 to September, 1974; and since then larger than about magnitude 1.5 in the site region (Richins, 1979). Small to moderate sized earthquakes are numerous in the site region and are associated with the Wasatch fault zone, Basin and Range faults such as those on the west flank of the Oquirrh Range, and events which cannot be assigned to known structures.

In the site area, a moderate sized earthquake (M = 5.2) occurred on September 2, 1962 (hypocenter located at latitude 40° 42.92'N, longitude 112° 5.33'W, depth of 7 kilometers - the average depth value of layer 2, a 5.9 km/sec. velocity layer extending from 1.4 to 15.5 kilometers in depth - Richins, 1979). The epicenter, in the northeast portion of Magna, was about 14.5 kilometers north of the site. Other smaller events are located near this area and are probably aftershocks of the M = 5.2 event. There is no geologically defined structure in the area of this seismicity.

5.4 Active Faulting

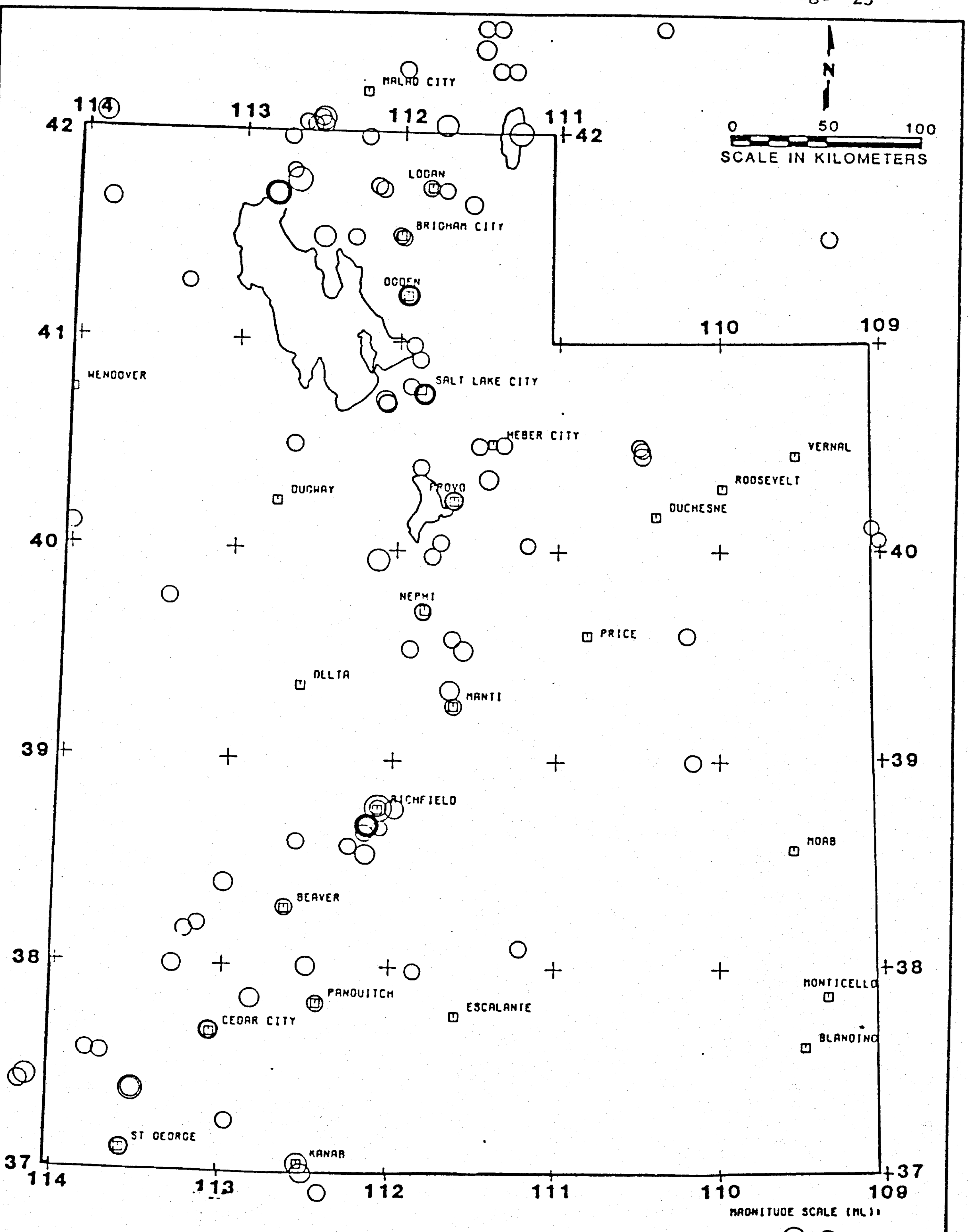
Various published fault maps indicate that active faulting (i.e. faulting in Holocene time - about the last 11,000 years) occurs at the surface in the site region (Bucknam, 1977; Anderson and Miller, 1979). Low-sun-angle (LSA) aerial reconnaissance and field reconnaissance in the site area has further delineated the active

FIGURE 2



(after Arabasz and others, 1979)

UTAH EARTHQUAKES: 1850 - 1949



(after Arabasz and others, 1979)

1850 - JUNE 1978, MAG 4.0 (INT V) OR GREATER

faults in the area. Two major fault systems with Holocene activity are within the site region, the Wasatch Front about 25 kilometers to the east, and the western frontal fault system of the Oquirrh Mountain, about 10 kilometers west of the site.

The Wasatch frontal fault system is well studied (e.g. Arabasz and others, 1979; Smith and others, 1979; Bucknam and others, 1980; Swan and others, 1980; Swan, 1983) and has been assigned a Maximum Credible Earthquake (MCE) of M = 7.6. Recurrence rate for earthquakes along the Wasatch Front have been determined for some segments of the fault zone as between 500 and 2,600 years (Swan and others, 1980). Arabasz and others (1979) estimate for the entire zone an M = 7.5 event occurs every 232 to 263 years. This is not necessarily the segment of the zone adjacent to the site region. Doser and Smith (1982) estimate an M = 6.5 to 7.5 event every 387 to 667 years on one of the segments using geologic moment rates. There have been no historical events with surface rupture on the Wasatch system.

Anderson and Miller (1979) have reported late Quaternary (10,000 to 500,000 years before present) faulting on the northwest flank of the Oquirrh Mountains. Low-sun-angle aerial reconnaissance accomplished for this study shows that the faulting occurs well below the elevation of Lake Bonneville shorelines and displaces the shorelines, indicating that the faulting is less than about 11,000 years old, thus, Holocene in age. This conclusion is confirmed by interpretation of 1:24,000 scale color infrared aerial photography of the area. The Holocene rupture occurs on a segment of the fault zone which extends from the north end of the Oquirrh Mountains south for a distance of about 23 miles (37 km). At the southern end, the fault is well segmented from the frontal fault system of the southwestern Oquirrh Mountains by a 90 degree change in trend and intersection of the range front by structures associated with South Mountain, the topographic division between the Tooele and Rush Valleys. The frontal fault of the southwestern Oquirrh Mountains is also an active fault, located 16 miles (26 km) from the site at its northern terminus. Wallace (1982) indicates recurrence intervals on Basin and Range faults such as these zones may exceed 10,000 years.

Seismicity in the Magna area suggests the possibility of active faulting; however, low-sun angle aerial reconnaissance and interpretation of 1:24,000 scale color infrared aerial photography did not reveal Holocene surface rupture in the area. At the site, well above Lake Bonneville shoreline elevations, older surfaces, probably early Pleistocene in age (0.5 to 1.8 million years before present), show no evidence of surface rupture. Wallace (1982) indicates that moderate to large earthquakes (M greater than about 6.0) are accompanied by surface rupture in the Basin and Range Province. The lack of surface rupture in the Magna area is indicative of no events larger than about M = 6 during the Holocene and at the site during the late Quaternary periods.

5.5 Seismic Zoning

Several seismic zoning studies have been accomplished in the site region. Algermissen and others (1982) indicate that maximum acceleration in rock expected at the site (as a percent of unit gravity) with a 90 percent probability of not being exceeded for ten years is 0.09 g, for fifty years 0.20 to 0.28 g, and for 250 years about 0.4 g.

Bucknam and others (1980) include the site in a region believed only to contain late Quaternary but pre-Holocene scarps. This classification may change as the area includes the Holocene faulting delineated in this study on the northwest front of the Oquirrh Mountains. Their study of this source (their number IIb) shows a recurrence interval in the source area of about 5,000 years for M = 7.0 to 7.6 earthquakes.

Ward (1981) has reviewed and revised the seismic zones of the 1979 Uniform Building Code. He places the site in an area of U-4 designation equal to a UBC-3 seismic zone but noting that UBC-3 seismic requirements must be fully complied with, including design review and field inspection, to ensure compliance.

5.6 Evaluation of Earthquake Design Parameters

Earthquake sources/source areas considered in this evaluation are listed in Table 1. These include the segment of the Wasatch fault zone east of the site, the northwest segment of the western frontal fault system of the Oquirrh Mountains, and a source area in the Magna area, defined from historical seismicity.

TABLE 1

DESIGN CRITERIA

Source/ Source Area	Maximum Length	Maximum Credible (1) Earthquake Magnitude	Distance to Site	Estimated Maximum (2) Acceleration in Rock at Site (Fraction of Unit Gravity)
Wasatch Zone		7.6 (3)	15 Miles (25 km)	0.38 g ± 0.06
Northwest Oquirrh Mountains Zone	24 Miles (39 km)	7.0	6 Miles (10 km)	0.45 g ± 0.06
Magna Source Area		5.2	8 Miles (13 km)	0.16 g ± 0.06
Southwest Oquirrh Mountains Zone	18 Miles (29 km)	6.8	15 Miles (24 km)	0.25 g ± 0.06

1) Calculated from Slemmons and others (1982), normal-slip fault relationship: $M = 0.809 + 1.341 (\log L)$, L = length of fault in meters.

2) Extrapolated from Seed and Idriss (1982).

3) From Bucknam and others (1980).

An MCE has been developed for these sources/source areas and the on-site maximum acceleration, in rock, has been calculated. The MCE's are calculated from regressions of Slemmons and others (1982) based on the rationale provided by Slemmons (1977) for the northwest Oquirrh Mountains fault system. MCE's used for the Wasatch frontal fault system developed by Bucknam and others (1980) are adopted for this study. The largest historical event which has occurred for the source area defined by historical seismicity in the Magna area is considered as the MCE.

Variations in the accelerations presented in Table 1 for the site can be expected as a result of local soil conditions at the site. Seed and Idriss (1982) report that for an acceleration at a site in rock of 0.5 g, the acceleration in stiff soils would be about 0.45 g, in deep cohesionless soils about 0.38 g, and in soft to medium stiff clay and sand about 0.28 g. The response over the site probably would be in the range of the stiff soils to deep cohesionless soil profiles over rock.

6. DISCUSSION & RECOMMENDATIONS FOR FOUNDATION SUPPORT SYSTEMS

Recommendations for foundation design of the conveyor system are presented in this section.

Fill was encountered in Borings C-1 through C-4 to depths ranging from 7.5 to 40 feet below the existing ground surface. The remainder of the ore conveyor corridor is underlain by a variable thickness of Harkers Allumium mantling bedrock. Due to design differences associated with the existing fills and alluvial deposits, foundation recommendations will be addressed in separate sections.

Shallow spread-type foundations bearing on undisturbed native soils or properly compacted structural fill are recommended for support of the overland portion of the conveyor system. Recommendations for shallow foundations are presented in Sections 6.2.1 and 6.3.1

Drilled cast-in-place concrete pier foundations provide an alternative system which may be an economical way to provide reliable support the elevated portions of the conveyor line, particularly at the railroad yard crossing near Bingham Canyon and along the final portion of the conveyor line as it rises 170 feet above grade to the coarse ore stockpile area located at the proposed concentrator site. Drilled piers have the advantages of more rapid construction and generally, greater load bearing capacity and lower settlements than shallow spread-type foundations. Recommendations for drilled piers are presented in Sections 6.2.2 and 6.3.2

6.1 Site Grading

Guide specifications for site grading are presented in Appendix D. These specifications outline the recommended surface preparation for areas in which fill will be placed (including subexcavation of softer surface soils), quality requirements for fill material and recommended degree of compaction of fills and the surface of the native soils.

Two categories of structural fill are included in the specifications. Class 1 fill consisting of selected silty or slightly clayey sand and gravel is to be placed in structural fills within the foundation areas, and the upper 2 feet of fill in roadways and yard areas. This fill can be obtained from the site, but selective excavation will be necessary. All excavated materials at the site are suitable for Class 2 fill. This material is to be placed in roadway and yard fills below 2 feet and in nonstructural fills.

Granular Base and Free Draining Backfill in significant quantities are not available in the excavated materials immediately within the project area without extensive processing and/or crushing of cobbles and boulders. It appears that a suitable source of these materials can be obtained from the "Harper Excavation" in the Coon/Harker Canyon area. It is anticipated that some processing of these materials may be required, however no borrow investigation or laboratory analysis has been conducted as part of this investigation to evaluate the required processing.

6.1.1 Topsoil Removal

It appears that normal stripping and grubbing operations will be sufficient to remove any detrimental organic material.

It is recommended that if fills, foundations or pavements are to be placed above the existing surface soils, a site specific observation of the area be made prior to placement of fills or concrete to verify the suitability of the subgrade soils.

6.1.2 Cut/Fill Slopes

Recommendations for cut and fill slopes for the grading of the project are as follows:

A. Permanent cut and fill slopes in soils should be no steeper than 1.5:1 (horizontal to vertical). Specific cut and fill slopes are presented in Section 7.1.

B. In general, temporary slopes for open cuts could be made at 1:1. Where trench shields are provided for the protection of workmen in trenches, temporary slopes in the range of 1/4:1 to 1/2:1 could be made. Temporary cut slopes should be inspected at least twice daily for soft zones, tension cracks at the top of slopes, and other evidence of deformations and necessary alterations in slopes be made accordingly.

6.1.3 Excavation Conditions

It is anticipated that most of the materials within the anticipated depths of excavation can be excavated with conventional earth moving equipment without the need for blasting. It is believed that most, if not all, the materials can be excavated with dozers and scrapers. Light to moderate "ripping" may be necessary for excavations in the deeper cemented soils and zones of soft rock. Blasting may be required in localized areas of competent bedrock which may be encountered in some of the deeper cuts along the conveyor line corridor. Utility trenching may be relatively difficult in some of the firmer soils encountered at the site.

Because of subexcavation requirements and the necessity for selective excavation and placement of fills, careful quality control of earthwork, under the direction of a qualified geotechnical engineer, is particularly important for this project.

6.2 Foundation Recommendations - Alluvial Deposits (Harkers Alluvium)

6.2.1 Shallow Foundations

6.2.1.1 Bearing Pressure

A safe net allowable soil bearing pressure of 5,000 psf should not be exceeded in the design of shallow spread-type foundations. This value applies to full dead plus design live loads and may be increased by one-third when considering wind or seismic forces.

These recommendations apply to foundations bearing on native soils or structural fill and are based on the site grading procedures given in Section 6.1.

Due to frost considerations, minimum foundation depths should be 4.0 feet below the lowest adjacent finished grade. Two feet and 1.33 feet are the minimum recommended widths of square and continuous foundations, respectively.

6.2.1.2 Upward Loads

The safe capacity of foundations subjected to upward loads should be calculated on the basis of the weight of the foundation and the weight of the soil within a prism bounded by lines projected outward from the top edge of foundations 30 degrees from the vertical. A soil density of 120 pcf is recommended for these computations. This is based on backfill around and above the footings being compacted in accordance with the recommendations presented in Section 6.1 and Appendix D. A factor of safety of at least 2.0 should be applied to ultimate capacities for long-term loads and 1.5 for short-term loads such as wind or seismic forces.

6.2.1.3 Lateral Loads

"Passive" soil resistance against the edges of footings, stem walls, etc., with backfill compacted in accordance with the guide specifications, should be considered as being equal to the forces exerted by a fluid of 350 pounds per cubic foot unit weight.

A coefficient of friction of 0.40 is recommended for computing lateral resistance between the bases of footings and slabs and the fill in analyzing lateral loads.

These values of equivalent fluid pressure for passive resistance and coefficient of friction should be considered ultimate values for purposes of design. However, they are low strain values which would be mobilized simultaneously at no more than about 1/2 inch deflection. They are intended for use in foundation design procedures given in the Uniform Building Code, (1985).

6.2.1.4 Estimated Settlements

Foundations along this portion of the conveyor line corridor will be underlain by predominantly firm native soils composed of clayey and silty sands and gravels. Settlements seated in these materials are expected to be essentially "elastic" in nature and occur very rapidly. A few layers or lenses of medium-stiff to stiff clays are present which may produce a significant element of longer-term settlements. Thus, both "immediate" and longer-term components of settlement were considered in the analysis.

Estimates of "immediate" settlements of foundations were made based upon the estimated relationship between E_s (soil Youngs Modulus) and depth given in Figure 4. These values were estimated from seismic refraction survey data, with correction for strain softening (Seed and Idriss, 1970), and from various correlations based on standard penetration resistance (Beckwith and Hansen, 1982; Martin, 1977). Elastic settlements were computed by the method presented by Carrier and Christian (1978), which includes a reduction factor of 0.59 to 0.71 for geometric effects and allows consideration of an incompressible boundary at depth.

Consolidation settlements were computed by conventional methods based on the results of one-dimensional consolidation tests obtained during our Geotechnical Investigation for the concentrator site. Consolidation test data indicates that the cohesive soils beneath the site are overconsolidated with the

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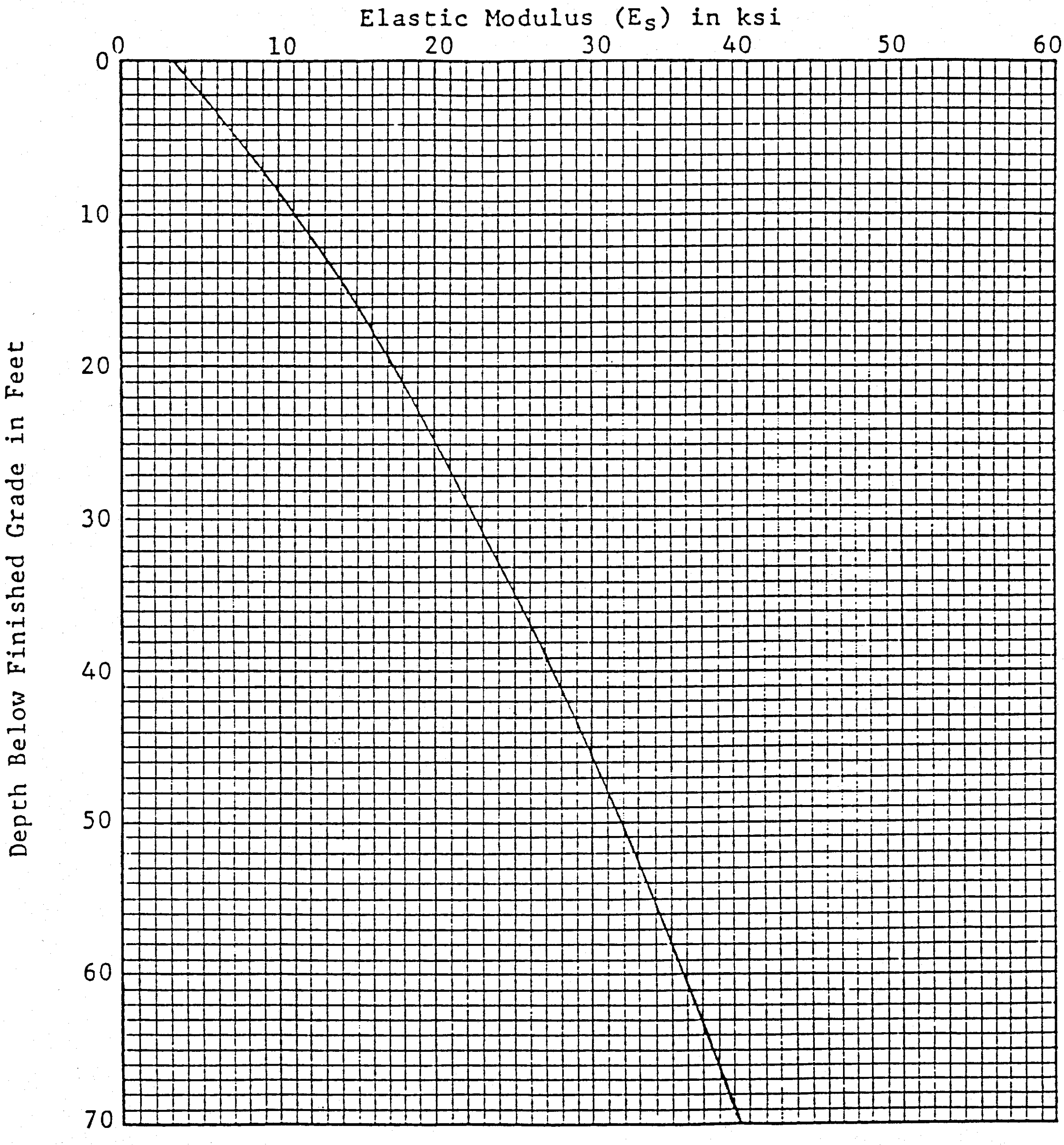
FIGURE 4

ESTIMATED ELASTIC MODULUS OF SOILS (E_s) FOR STATIC LOADING CONDITIONS AS A FUNCTION OF DEPTH



average preconsolidation ratio of the soils of about 2.5 being estimated. This is at least partly due to past erosion of a portion of the Harkers Alluvium from the site. Cycles of desiccation of the clayey soils may have also contributed to preconsolidation. It was estimated in calculations that 10 percent of the soil strata were subject to significant longer-term settlements. A correction factor of 0.5 was applied to these settlements to allow for overconsolidation.

It is estimated that settlements of foundations designed in accordance with the criteria given above will not exceed 3/4 inch. It is expectd that, in most cases, settlements will be on the order of 1/4 inch. These estimates apply to existing moisture contents in the native soils and compaction moisture contents in structural fill. Substantial moisture increases could create some additional settlements.

6.2.1.5 Elastic Constants for Dynamic Analysis

An estimated range of elastic constants for use in dynamic foundation design is presented in Figure 5. These parameters have been estimated on the basis of the seismic refraction surveys and correlations with standard penetration test (SPT) data and relative density in granular soils, and the relationship between relative density and shear wave velocity presented by Seed and Idriss (1970), Hardin and Drnevich (1972) and Hardin and Richart (1963).

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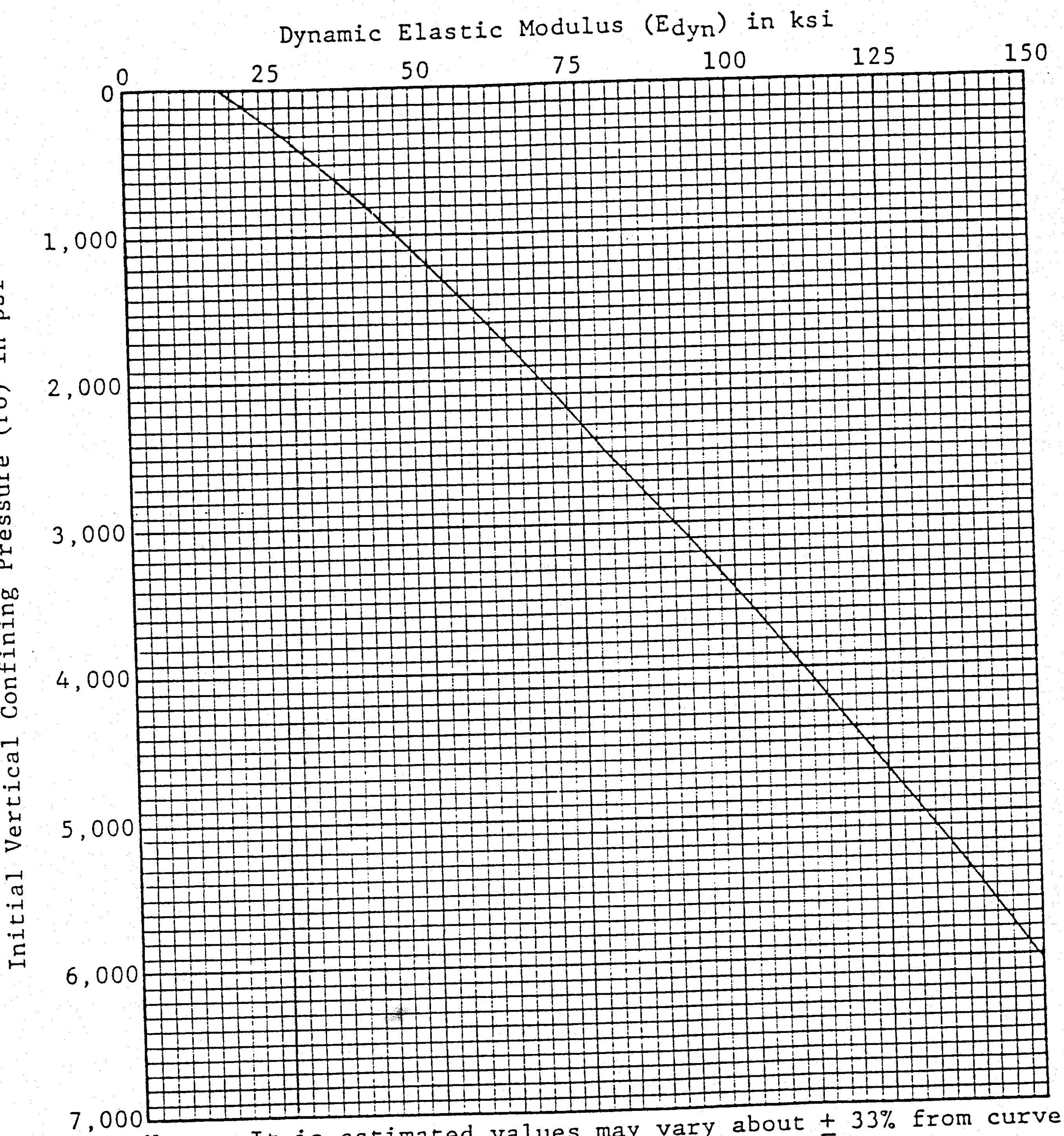
FIGURE 5

ESTIMATED DYNAMIC ELASTIC MODULUS AS A FUNCTION OF INITIAL VERTICAL CONFINING PRESSURE



Note: It is estimated values may vary about ± 33% from curve at various locations

Figure 5 gives a range of estimated dynamic elastic modulus Edyn versus initial vertical confining pressure (Po).

For purposes of determining Po, it is assumed that the "average point" of stress created by the footing is 3/4a after a procedure given by Lambe and Whitman (1969) (see Figure 6). Po should be calculated using the following equation:

$$Po = \gamma z + Ip\,(q)$$

where: γ = 120 pcf = the unit weight of the soil.

z = 3/4a = distance from the base of footing to the "average point" of stress in feet.

Ip = stress influence coefficient.

q = static bearing pressure of footing before dynamic loads are applied in pounds per square foot. To include weight of concrete in foundation below finished grade.

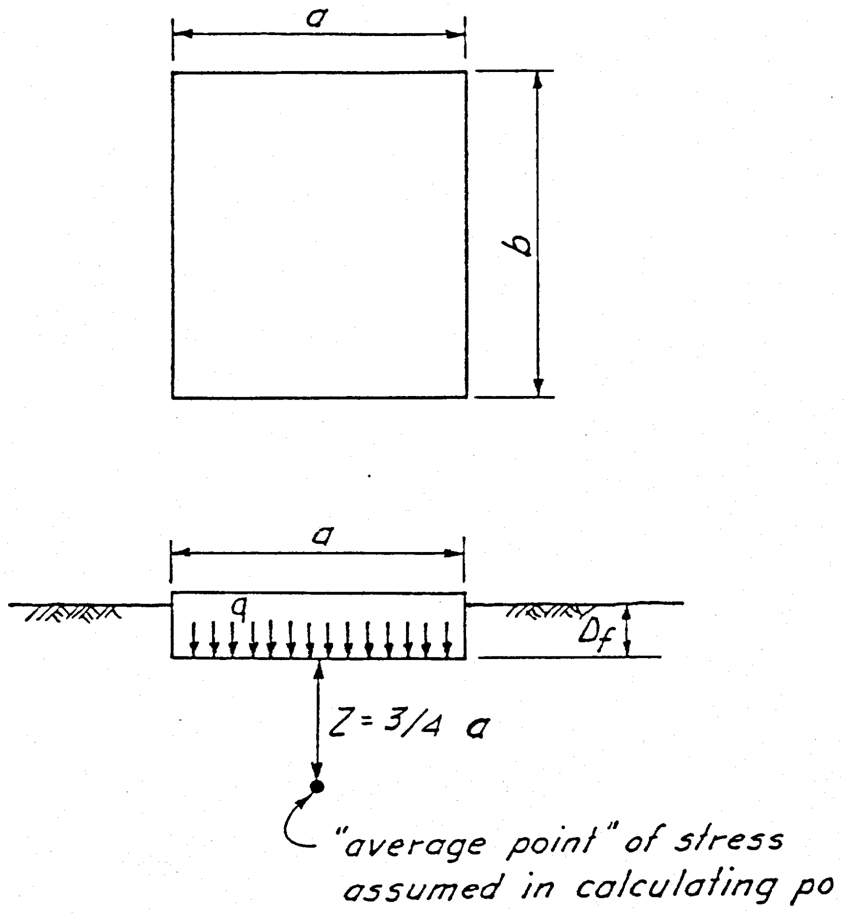
Values of Ip for various ratios of footing width, a, to footing length, b, are as follows:

a/b	Ip
1.0	0.36
1.5	0.43
2.0	0.47
3.0	0.51

The values of Edyn given in Figure 5 are based upon shear strains of about 10^{-4} percent. Shear strains are generally near or below 10^{-4} percent

FIGURE 6

RECTANGULAR FOOTING SUBJECTED TO DYNAMIC LOADS



for machine foundation design analysis. However, if higher strains should occur, Edyn should be reduced for "strain softening" effects of the basis of relationships given by Seed and Idriss (1970).

A Poisson's ratio value (μ) in the range of 0.15 to 0.30 is also recommended for design.

6.2.2 Drilled Piers

Geotechnical conditions are favorable for the construction of drilled, cast-in-place concrete piers. Little caving or sloughing is anticipated. It is estimated that concrete volumes would generally be on the order of 10 to 15 percent above the neat volumes indicated by the plans. The presence of cobbles and occasional boulders may cause zones of difficult drilling, but should not preclude the use of drilled piers.

However, if drilled foundations are selected for any part of the project, geotechnical conditions for construction should be confirmed by at least two large diameter holes to the maximum foundation depth. Optimum locations for the test holes can be established by the geotechnical engineer.

Straight drilled machine-cleaned foundations appear to be one of the best systems for certain elements of the project such as the elevated portions of the conveyor. Recommendations for this type of system are given in the following sections.

6.2.2.1 Vertical Capacities

Minimum depth should be 20.0 feet below finished grade. Safe capacities for downward and upward loads are shown on Figures 7 and 8. The charts show the relationship between safe capacities versus depth of penetration below finished grade and are based on a factor of safety of 2.5.

The estimated capacities apply to full dead plus design live loads and may be safely increased by one-third for total loads, including wind or seismic forces. The capacities apply to the allowable soil supporting capacity and do not consider the structural strength of the piers.

Criteria presented in this and the following section applies to isolated piers spaced no closer than 3 diameters on center perpendicular to the line of thrust and 6 diameters on center parallel to the line of thrust. Group reduction factors can be supplied once pier spacing is determined, if pier groups are required.

6.2.2.2 Resistance to Lateral Loads

The ultimate lateral bearing pressure versus depth relationship for various diameters of piers was computed using the method by Hansen (1961), and is presented in Figure 9.

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FIGURE 7

RECOMMENDED SAFE DOWNWARD CAPACITIES OF STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE PIERS

(HARKERS ALLUVIUM)

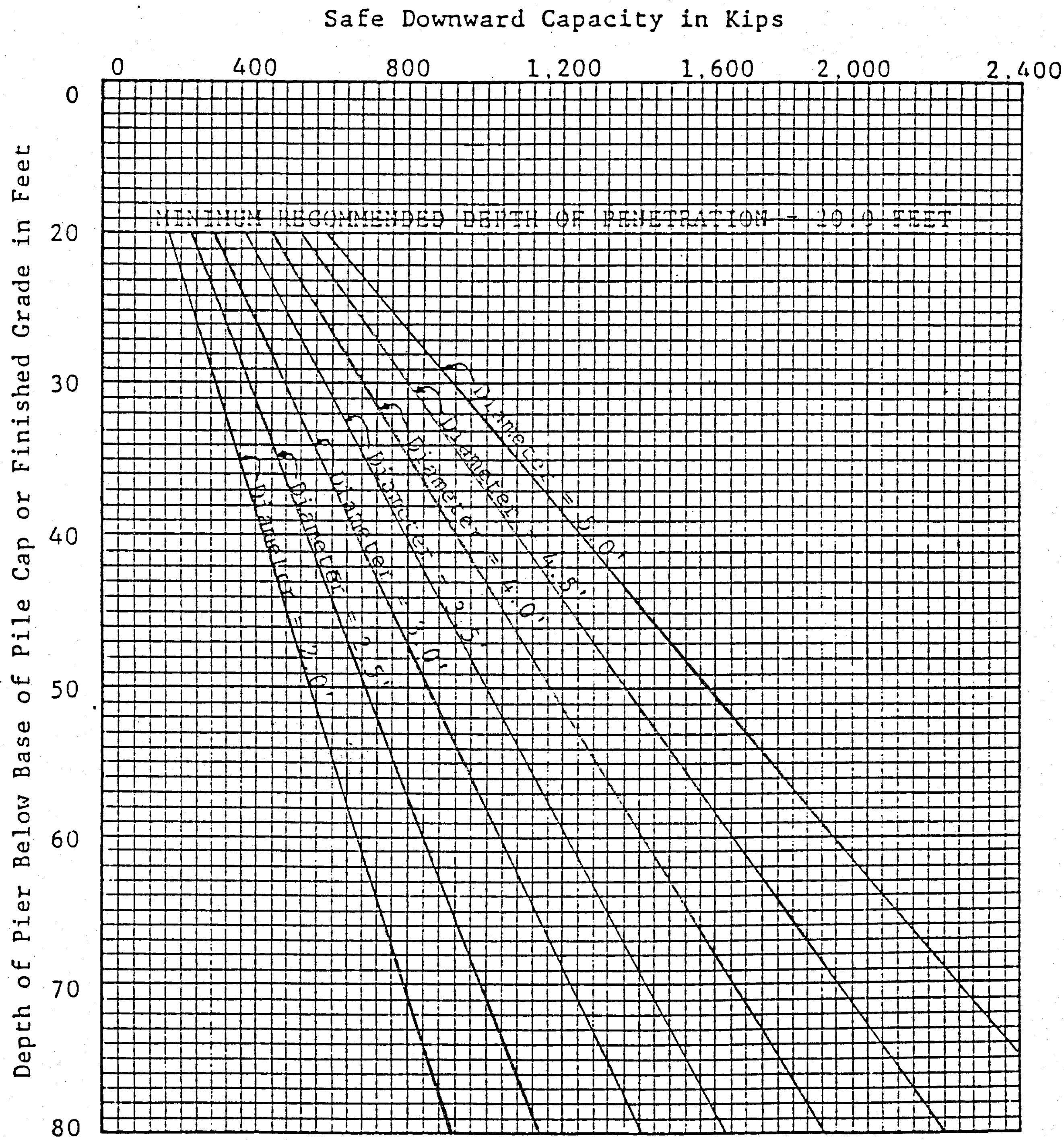

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FIGURE 8

RECOMMENDED SAFE UPWARD CAPACITIES OF STRAIGHT, DRILLED CAST-IN-PLACE CONCRETE PIERS

(HARKERS ALLUVIUM)

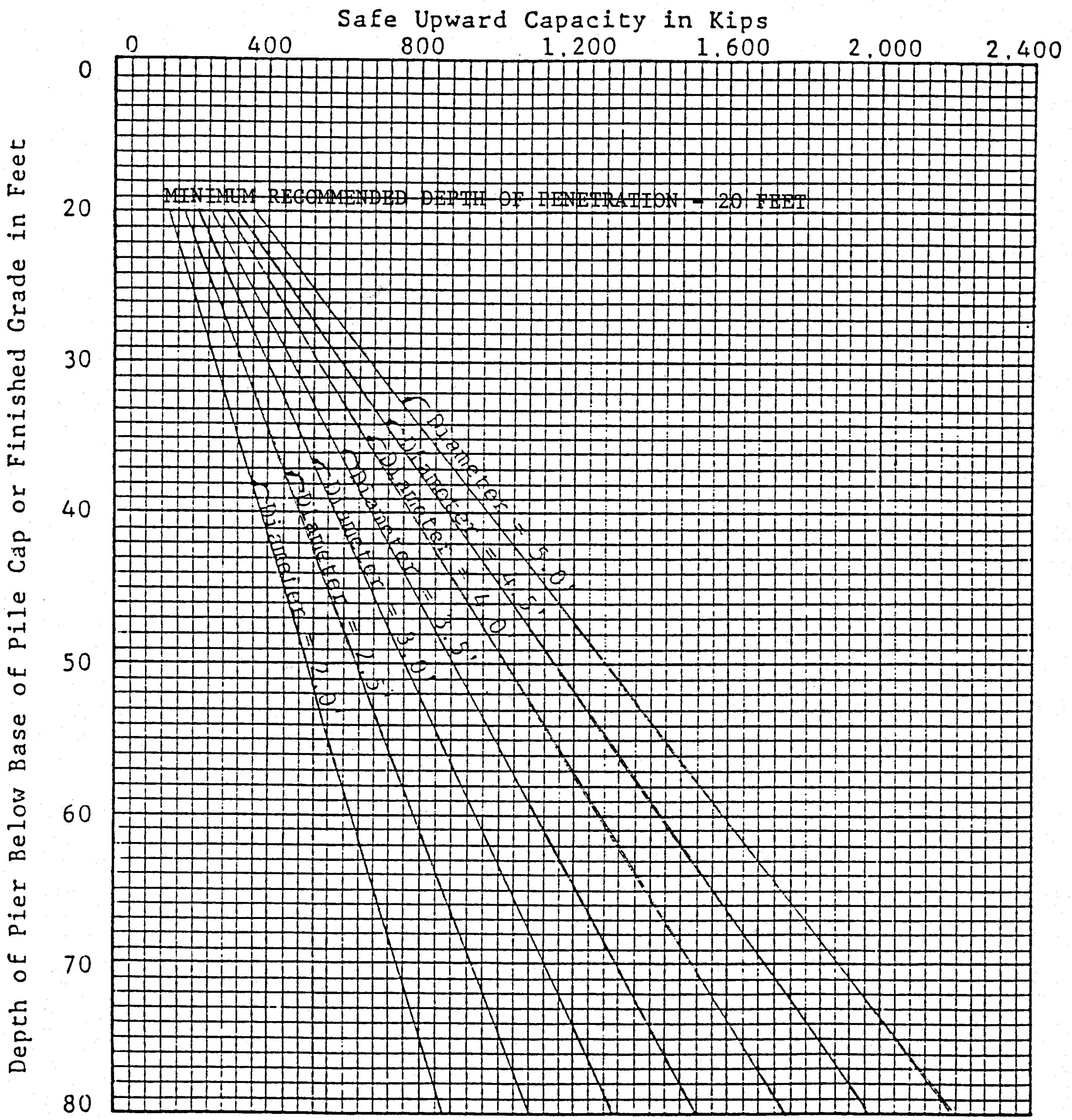

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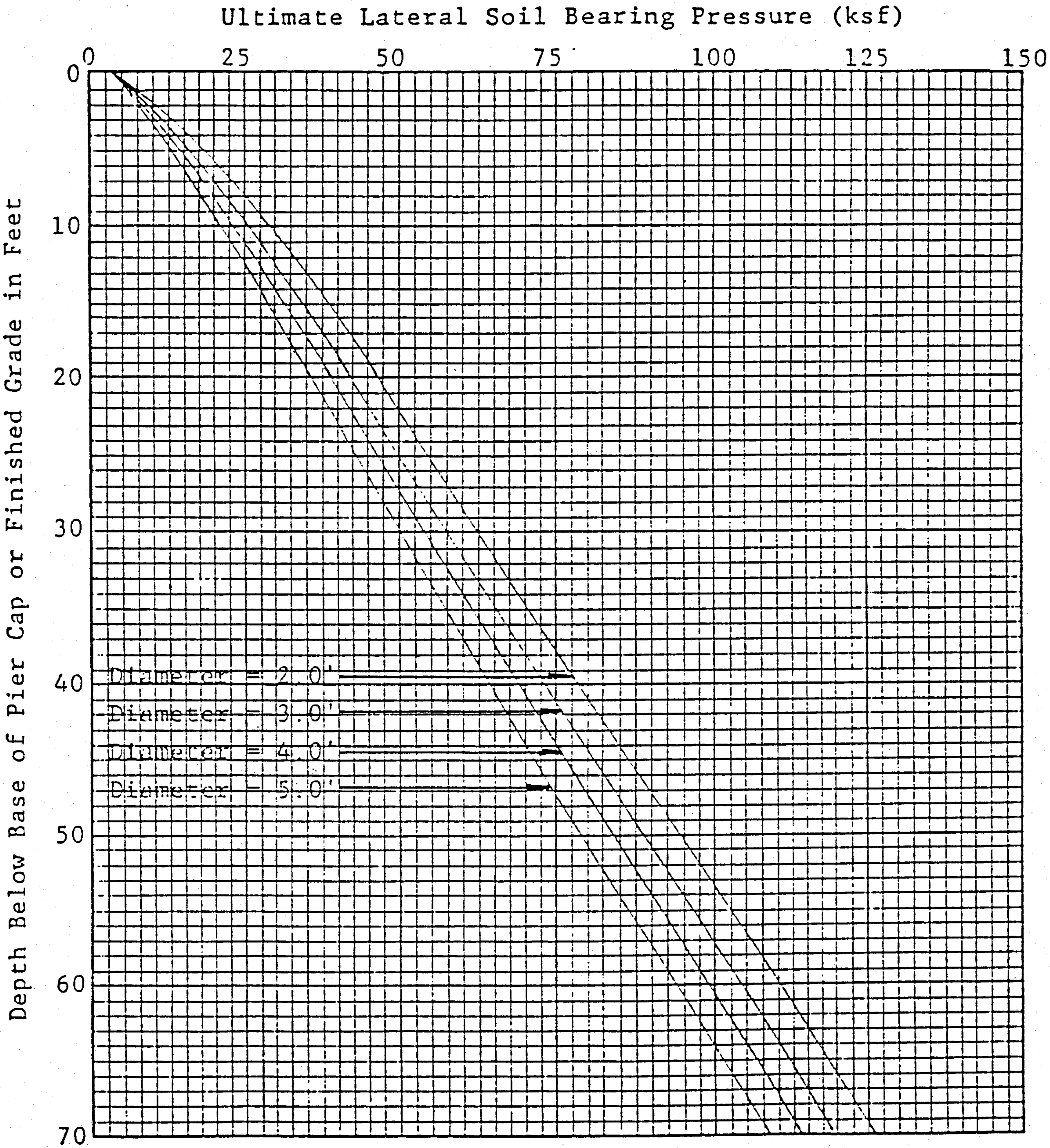
FIGURE 9

ULTIMATE LATERAL SOIL PRESSURE FOR STRAIGHT CAST-IN-PLACE CONCRETE PIERS
(HARKERS ALLUVIUM)



Methods based on theory of beam-on-an-elastic half-space are recommended for computation of soil reactions and pier deflections, moments and shears at design loads. These should be considered first order estimates. Depending on analysis results, the p-y curve method can be utilized to provide a better assessment of these quantities. We have an in-house computer capability to perform this level of analysis, and would be available to do so, once applied lateral loads and moments have been established.

6.2.2.3 Estimated Settlements

It is estimated that settlements of isolated pier foundations supporting column loads up to 850 kips will not exceed about 1/4 inch.

6.2.2.4 Positional Tolerances

All drilled piers should be installed so that the centerline of the top of the pier is within 3 inches of the plan location. Vertical piers with diameters less than 3 feet should deviate from plumb no more than 1 percent of the pier length. A deviation of no more than 2 percent of the length should be allowed for piers of 3 feet or more in diameter.

Pier excavations exceeding the specified tolerances should be redrilled to a larger diameter which can be installed in conformance with the required tolerances.

6.2.2.5 Placement of Concrete

Concrete should be placed through a hopper or other device approved by the geotechnical engineer so that it is channeled in such a manner to free fall and clear the walls of the excavation and reinforcing steel until it strikes the bottom. The spacing of vertical reinforcing bars should not be less than 6 inches and the spacing of horizontal ties should not be less than 12 inches. A minimum of 6 inches of concrete cover should be maintained around the reinforcing steel. A discharge pipe, a minimum of 8 inches in diameter, extending to near the bottom of the reinforcing steel may be necessary for proper concrete placement. Adequate compaction will be achieved by free fall of the concrete up to the top 10 feet. The top 10 feet of concrete should be vibrated to achieve proper compaction. Concrete slump during placement should be in the range of 4 to 6 inches. The concrete mix should be designed, from a strength standpoint, so it can be placed in this range of consistency.

6.2.2.6 Observations of Pier Construction

Continuous observation of the construction of drilled piers should be carried out under the direction of a qualified geotechnical engineer and should be made by qualified engineering technicians or engineering personnel. The observations should verify proper diameter, depth, plumbness and cleaning, and should also verify the nature of the materials encountered in the pier excavations. Concrete placement should be continuously observed to verify that it meets requirements. The use of mirrors and/ or intense lights will be necessary for proper observations. A light water spraying of the holes for dust control may be necessary. With these techniques, the machine cleaned excavations can be properly observed from the surface.

6.2.2.7 Cleaning of Drilled Pier Excavations

Straight, drilled pier excavations should be advanced with single flight auger to the required penetration. Machine cleaning should then be verified by measurements to confirm that the excavations are open to the required depths. The auger should then be placed back in the holes and two additional passes made to clean loose materials from the bottoms of the excavations. Specifications should be written in such a manner that the contractor may be required to introduce small amounts of water into the excavations to aid in the cleaning as directed by the geotechnical engineer.

6.3 Foundation Recommendations - Existing Fills

6.3.1 Analysis and Discussion

The existing fill along the conveyor alignment as encountered in the exploratory borings, is typically comprised of relatively firm materials but did contain some deleterious materials such as railroad ties, wire, concrete debris and slag. While the subsurface conditions encountered in the borings indicate the materials will provide adequate support for spread footings on drilled piers designed and constructed in accordance with the recommendations presented in the following sections, it is possible that there are localized areas of less competent fill materials or with excessive quantities of organic material. It should be recognized that there may be potential for settlements in excess of those estimated herein as a result of these potential subsurface conditions. Alternates which would minimize this potential could include extending foundations through the fill to bear on competent native soil or the excavation and replacement of the existing fill with controlled structural fill. These approaches would appear to be cost probabilities in general.

6.3.2 Shallow Foundations

6.3.2.1 Bearing Pressure

A safe net allowable soil bearing pressure of 5,000 psf should not be exceeded in the design of shallow spread-type foundations. This value applies to full dead plus design live loads and may be increased by one-third when considering wind or seismic forces.

These recommendations apply to foundations bearing on existing fills or properly placed structural fill and are based on the site grading procedures given in Section 6.1. It is recommended that the footing excavations be observed by a qualified geotechnical engineer to verify suitable materials are exposed at the bottom of footing elevation.

Due to frost considerations, minimum foundation depths should be 4.0 feet below the lowest adjacent finished grade. Two feet and 1.33 feet are the minimum recommended widths of square and continuous foundations, respectively.

6.3.2.2 Upward Loads

The safe capacity of foundations subjected to upward loads should be calculated on the basis of the weight of the foundation and the weight of the soil within a prism bounded by lines projected outward from the top edge of foundations 30 degrees from the vertical. A soil density of 120 pcf is recommended for these computations. This is based on backfill around and above the footings being compacted in accordance with the recommendations presented in Section 6.1 and Appendix D. A factor of safety of at least 2.0 should be applied to ultimate capacities for long-term loads and 1.5 for short-term loads such as wind or seismic forces.

6.3.2.3 Lateral Loads

"Passive" soil resistance against the edges of footings, stem walls, etc., with backfill compacted in accordance with the guide specifications, should be considered as being equal to the forces exerted by a fluid of 350 pounds per cubic foot unit weight.

A coefficient of friction of 0.40 is recommended for computing lateral resistance between the bases of footings and slabs and the fill in analyzing lateral loads.

These values of equivalent fluid pressure for passive resistance and coefficient of friction should be considered ultimate values for purposes of design. However, they are low strain values which would be mobilized simultaneously at no more than about 1/2 inch deflection. They are intended for use in foundation design procedures given in the Uniform Building Code, (1985).

6.3.2.4 Estimated Settlements

Foundations along this portion of the conveyor line corridor will be underlain by existing fills consisting of predominantly firm, clayey and silty sands and gravels. Settlements seated in these materials are expected to be essentially "elastic" in nature and occur very rapidly. A few layers or lenses of medium stiff to stiff clays are present which may produce an element of longer-term settlement. Thus, both "immediate" and longer-term components of settlement were considered in the analysis.

It is estimated that settlements of foundations designed in accordance with the criteria given above will not exceed 1 inch. It is expected that, in most cases, a majority of this settlement will occur during construction. These estimates apply to existing moisture contents in the existing fills and compaction moisture content in structural fill. Substantial moisture increases could create some additional settlements.

6.3.3 Drilled Piers

Drilled, cast-in-place concrete piers appear to be one of the best systems for support of the elevated portions of the conveyor line. Apparent leachate fluids were encountered in the existing fills (Borings C-1, C-2 and C-3). Based on the granular nature of the fills and the results of the field investigation, caving or sloughing particularly below the phreatic surface should be anticipated. As previously noted, the drilled piers will be advanced through existing manmade fill containing some deleterious materials which could affect drilling operations as well as the design pier capacity. Should conditions be encountered which differ significantly from those found in the exploratory borings it may be necessary to alter the design pier depth.

Recommendations for drilled pier foundations are presented in the following sections.

6.3.3.1 Vertical Capacities

Minimum depth should be 20.0 feet below finished grade. Safe capacities for downward and upward loads are shown on Figures 10 and 11. The charts show the relationship between safe capacities versus depth of penetration below finished grade and are based on a factor of safety of 2.5.

The estimated capacities apply to full dead plus design live loads and may be safely increased by one-third for total loads, including wind or seismic

FIGURE 10

RECOMMENDED SAFE DOWNWARD CAPACITIES OF STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE PIERS IN EXISTING FILL

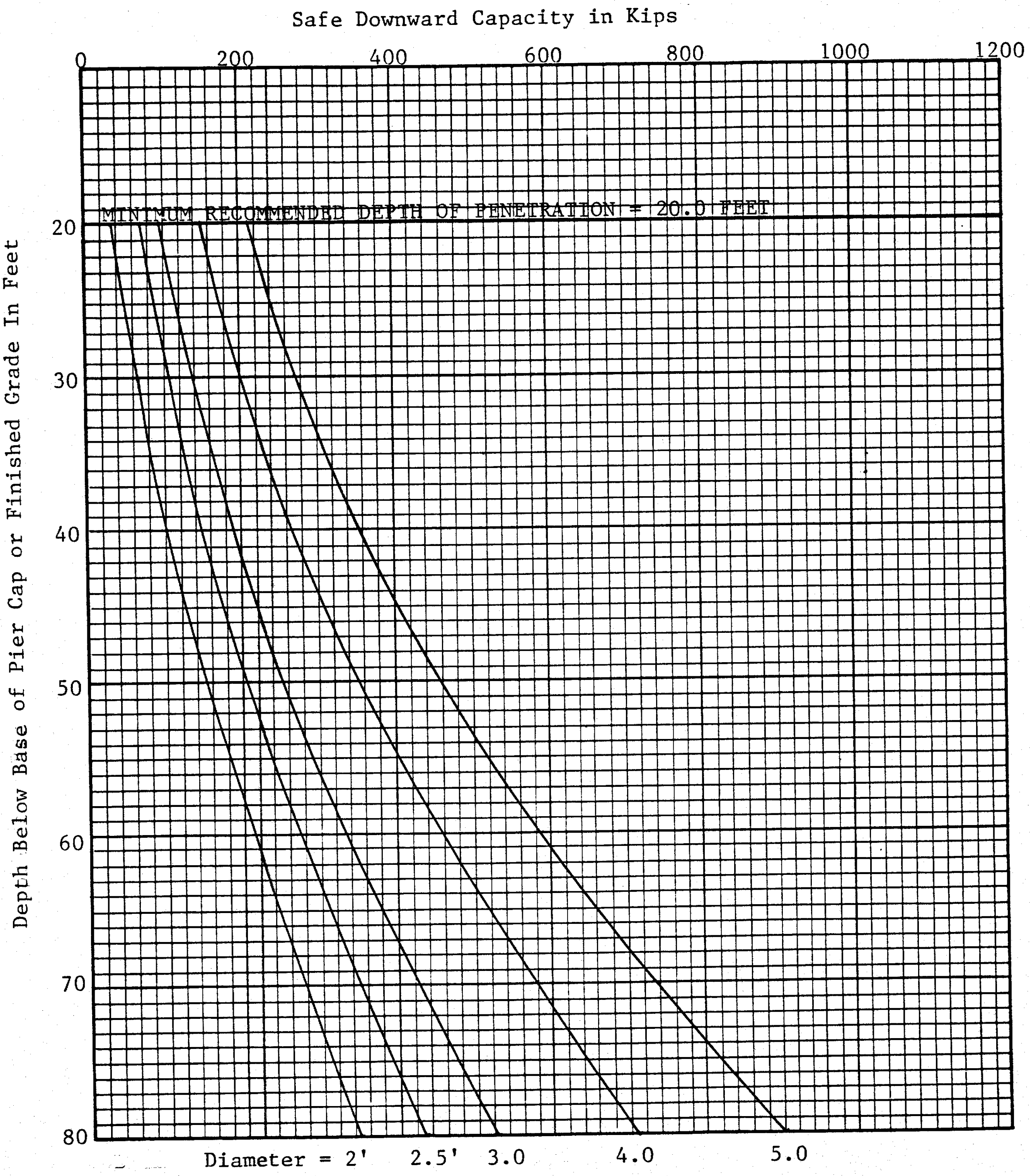
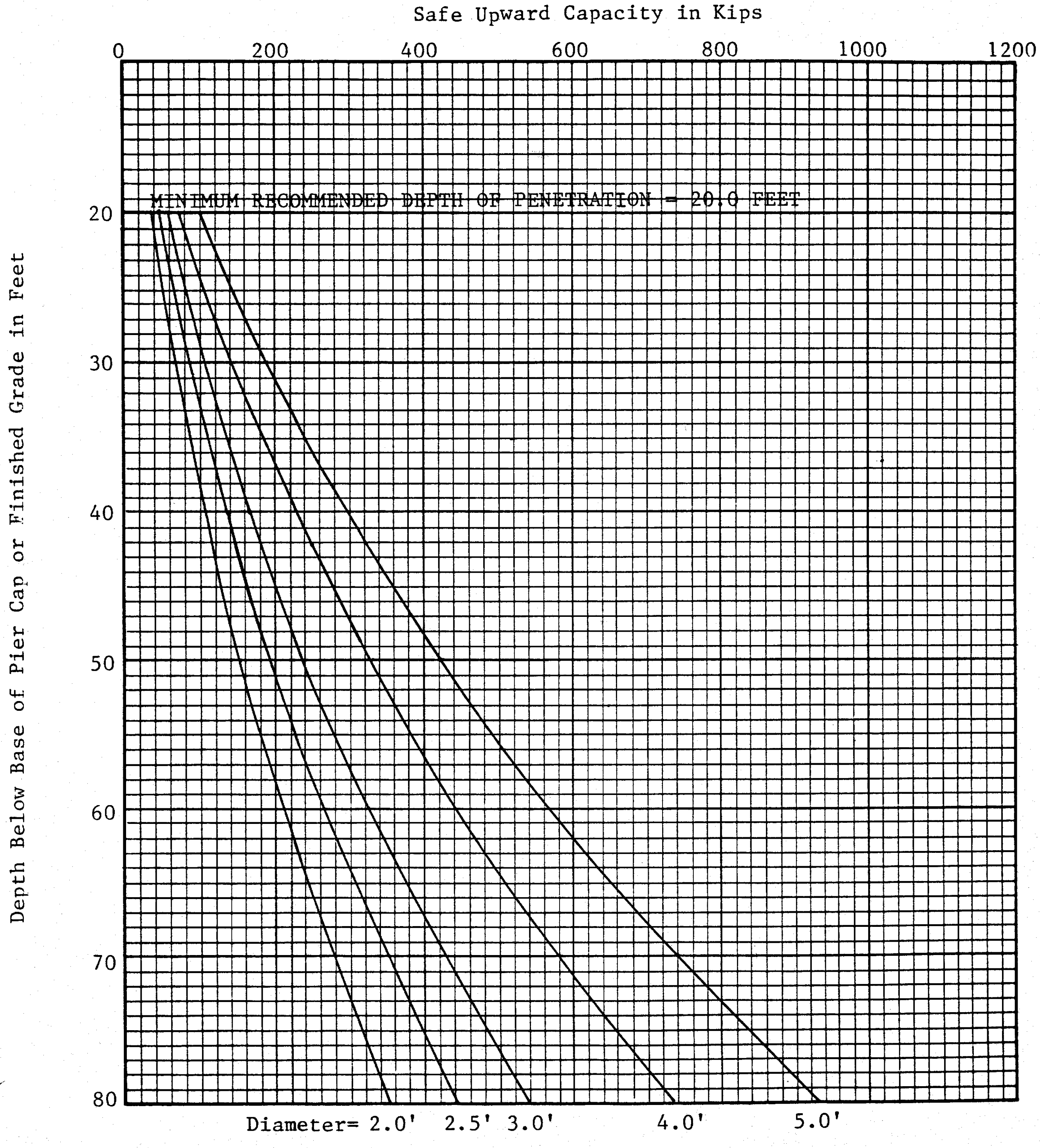


FIGURE 11

RECOMMENDED SAFE UPWARD CAPACITIES OF STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE PIERS IN EXISTING FILL



forces. The capacities apply to the allowable soil supporting capacity and do not consider the structural strength of the piers.

Criteria presented in this and the following sections applies to isolated piers spaced no closer than 3 diameters on center perpendicular to the line of thrust and 6 diameters on center parallel to the line of thrust. Group reduction factors can be supplied once pier spacing is determined, if they are required.

6.3.3.2 Resistance to Lateral Loads

The ultimate lateral bearing pressure versus depth relationship for various diameters of piers was computed using the method by Hansen (1961), and is presented in Figure 12.

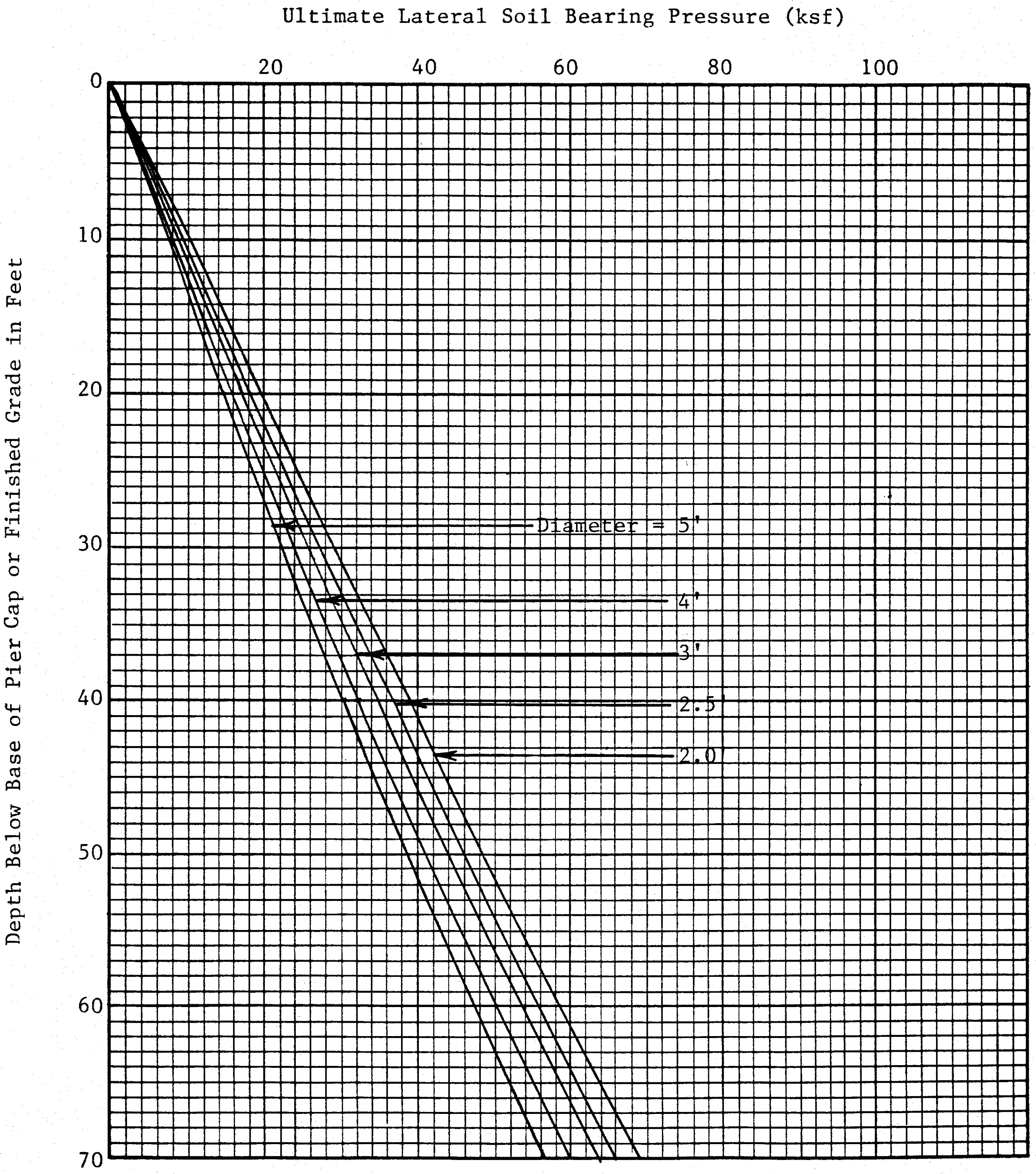
Methods based on theory of beam-on-an-elastic half-space are recommended for computation of soil reactions and pier deflections, moments and shears at design loads. These should be considered initial estimates. Depending on analysis results, the p-y curve method can be utilized to provide a better assessment of these quantities. We have an in-house computer capability to perform this level of analysis, and would be available to do so, once applied laterial loads and moments have been developed.

6.3.3.3 Estimated Settlements

It is estimated that settlements of isolated pier foundations supporting column loads up to 850 kips will not exceed about 1/2 inch.

FIGURE 12

ULTIMATE LATERAL SOIL PRESSURE FOR STRAIGHT, CAST-IN-PLACE CONCRETE PIERS IN EXISTING FILL



6.3.3.4 Positional Tolerances

All drilled piers should be installed so that the centerline of the top of the pier is within 3 inches of the plan location. Vertical piers with diameters less than 3 feet should deviate from plumb no more than 1 percent of the pier length. A deviation of no more than 2 percent of the length should be allowed for piers of 3 feet or more in diameter. Pier excavations exceeding the specified tolerances should be redrilled to a larger diameter which can be installed in conformance with the required tolerances.

6.3.3.5 Excavation of Drilled Piers

Due to the granular nature of the subsoils at depth and the presence of a perched groundwater table, caving or sloughing should be anticipated. The drilled pier excavations should either be cased or excavated utilizing slurry drilling techniques to prevent caving and/or squeezing of open excavations. If casing is chosen, a positive head must be maintained in the hole below the water table to prevent a "quick" condition from developing at the base of the excavations.

Straight, drilled pier excavations should be advanced with single flight auger or bucket auger to the required penetration. Machine cleaning should then be verified by measurements to confirm that the excavations are open to the required depths. The auger should then be placed back in the holes and two additional passes made to clean loose materials from the bottoms of the excavations.

6.3.3.6 Placement of Concrete

Concrete should be placed by means of a gravity feed tremie pipe or a combination concrete pump and tremie pipe. The tremie pipe should be a rigid, watertight pipe for the full length of the pier. It should not be less than 6 inches in diameter if concrete is pumped, or not less than 8 inches in diameter if gravity feed procedures are used.

The pipe should be equipped with a bottom valve, or other approved device which would prevent mixing of any water on slurry within the excavation with the concrete inside the pipe, and which would prevent the intrusion of slurry into the concrete in the event the tremie pipe has to be removed and replaced.

Reinforcing steel should be in place and the tremie pipe should be inserted to the bottom of the hole prior to concrete placement. The spacing of vertical reinforcing bars should not be less than 6 inches and the spacing of horizontal ties should not be less than 12 inches. A minimum of 6 inches of concrete cover should be maintained around the reinforcing steel. Concrete should be placed in a continuous operation in such a manner that the concrete always flows upward within the hole displacing fluid. The delivery pipe (and casing if utilized) should be withdrawn as the elevation of the concrete in the hole rises. The discharge end of the pipe should, at all times, be maintained at least 5.0 feet below the surface of the concrete. Raising of the tremie pipe should be done only when the pipe contains a sufficient head of concrete to prevent the formation of a void at the tip.

During concrete placement, the contractor should provide and maintain marking on the tremie pipe, a sounding device, or other adequate method with which to determine the relative elevation of the concrete surface and the end of the tremie pipe. Concrete slump during placement should be in the range of 6 to 8 inches. The concrete mix should be designed, from a strength standpoint, so it can be placed in this range of consistency. The top 10 feet of concrete should be vibrated to achieve proper compaction.

6.3.3.7 Observations of Pier Construction

Continuous observation of the construction of drilled piers should be carried out under the direction of a qualified geotechnical engineer and should be made by qualified engineering technicians or engineering personnel. The observations should verify proper diameter, depth, plumbness and cleaning, and should also verify the nature of the materials encountered in the pier excavations. Concrete placement should be continuously observed to insure that it meets requirements. A report should be submitted on each pier stating, in writing, that all construction operations have been observed and meet project requirements.

6.4 Seismic Design Criteria

Based on the seismic hazard analysis outlined in Section 5, it is recommended that the provisions of Zone 3 of the Uniform Building Code (International Conference of Building Officials, 1985) be used in the design of the basic structural systems. A characteristic site period Ts, of 0.2 seconds is recommended for use with the UBC design methods.

Although the seismic risk of the site is consistent with Zone 3, large earthquakes should be expected to occur near the site at long return intervals generating very intense ground shaking. Thus, for elements of the project where interruption of service would create serious problems or where damage to equipment would involve unusually large costs, special procedures such as those outlined by McBean and others (1983) may merit consideration for use. These procedures might apply to critical elements of the facility including elevated structural elements , sensitive processing equipment, tanks, machinery, electrical equipment, piping, and similar features.

In order to enable the selection of earthquake design parameters on the basis of risk analysis, the estimated relationship between effective peak horizontal ground acceleration (Aa), horizontal velocity (Av) and average recurrence interval are given in Figure 13. These curves are based on the studies of Algermissen and others (1982) and Hays and others (1981). Acceleration response spectra for various values of Aa for 5 percent damping are presented in Figure 14. The shape of these

FIGURE 13 Horizontal Velocity (A_v) and Effective Peak Horizontal Acceleration (A_a) Versus Return Period

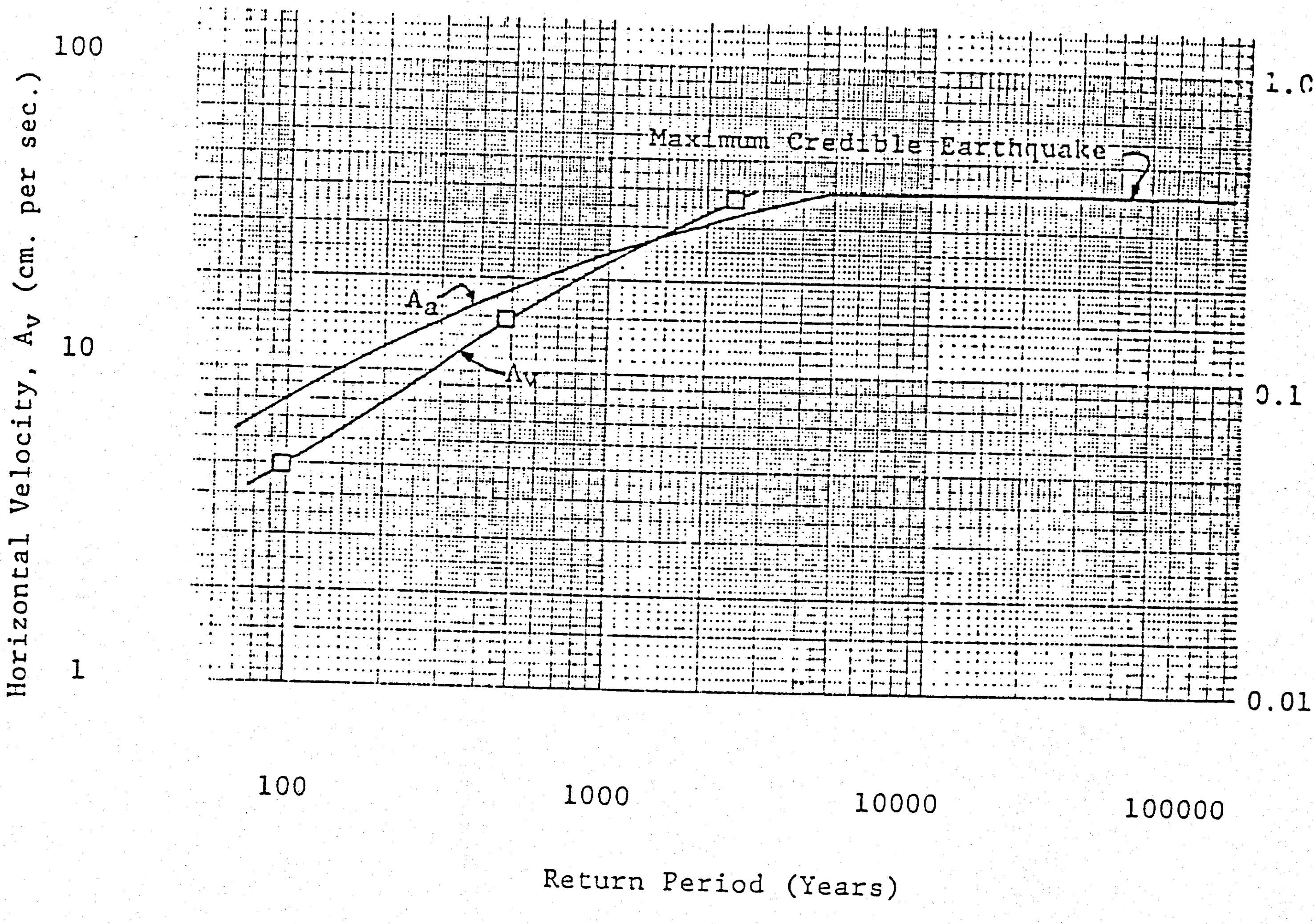


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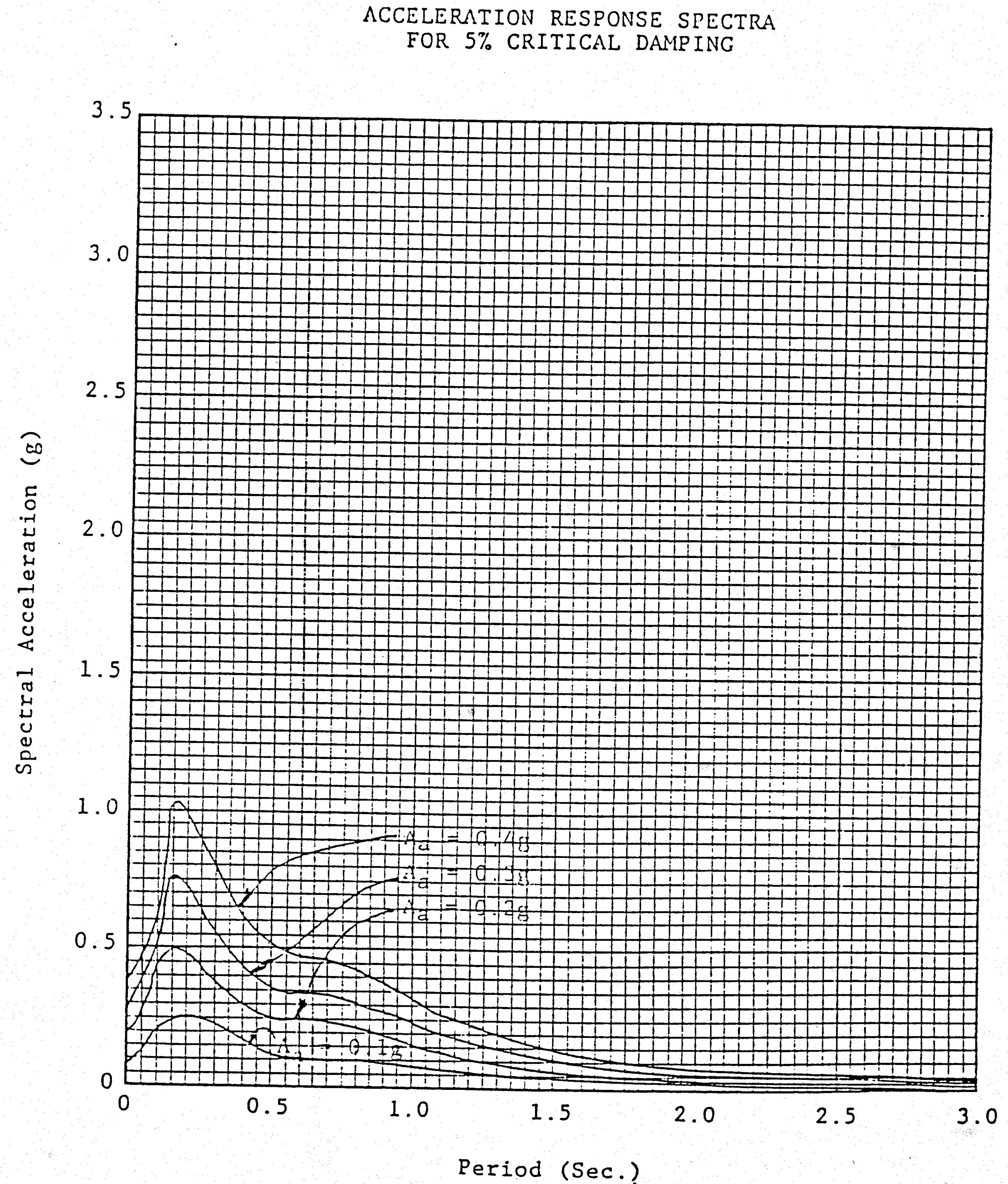
FIGURE 14



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curves was selected based on the analysis of average spectra measured during earthquakes for sites on rock and stiff soils reported by Seed and others (1976) and Mohraz (1976).

Vertical acceleration should be assumed to be 80 percent of horizontal acceleration.

6.5 Concrete Floor Slabs Cast-on-Grade

The use of a granular base course is recommended beneath all concrete floor slabs cast-on-grade. At least 6 inches of granular base should be placed beneath heavily loaded, industrial-type slabs, and at least 4 inches of granular base should be placed beneath lightly loaded slabs for office areas and areas of similar occupancy. Granular base should meet requirements given in Appendix D.

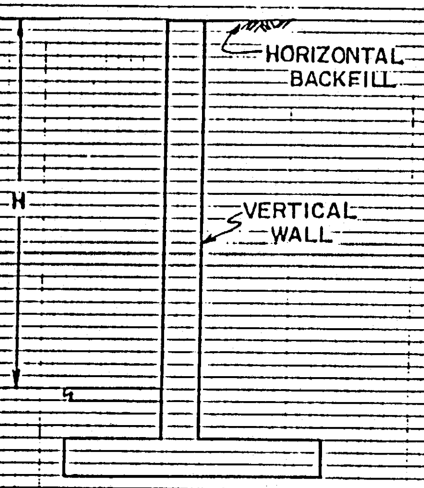
With the recommended site grading and granular base thicknesses, a modulus of subgrade reaction (k) of 300 pci is recommended for the structural design of concrete floor slabs cast-on-grade.

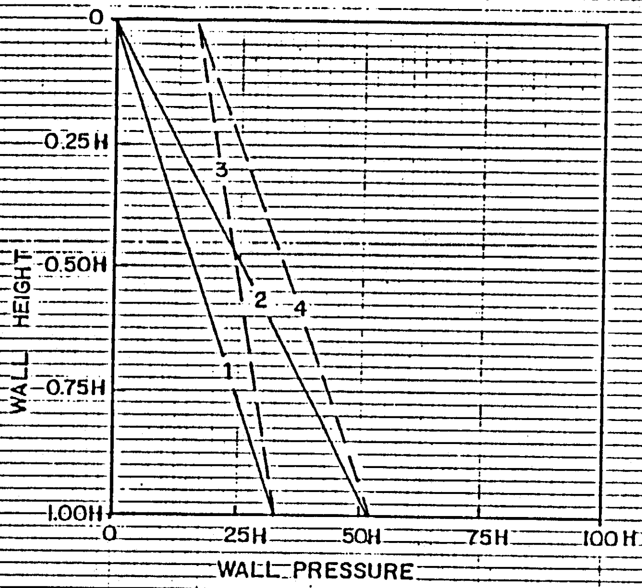
6.6 Retaining Walls

Rigid retaining walls free to rotate at the top will be subject to "active" earth pressures, while walls restrained from movement at the top will be subjected to "at rest" earth pressures. Recommended earth pressures for these two conditions for the case of static loads only and the case of seismic loads are presented in Figure 15. Seismic earth pressure diagrams for Aa = 0.18

FIGURE 15

RECOMMENDED EARTH PRESSURES AGAINST
BASEMENT WALL & RETAINING WALL





CASES SHOWN

1. ACTIVE EARTH PRESSURE
2. AT-REST EARTH PRESSURE
3. SEISMIC EARTH PRESSURE FOR ACTIVE CASE
4. SEISMIC EARTH PRESSURE FOR AT-REST CASE

BASIS OF COMPUTATION

$\phi = 36°$
$C = 0$
$\gamma = 125$ pcf
$\delta = 2/3\ \phi$
$A_a = 0.18g$*

* (Based on a return period of 500 years)

NOTATION

H = WALL HEIGHT IN FEET
ϕ = BACKFILL FRICTION ANGLE
δ = WALL-BACKFILL INTERFACE FRICTION ANGLE
C = BACKFILL COHESION
γ = BACKFILL UNIT WEIGHT
Aa = EFFECTIVE PEAK HORIZONTAL ACCELERATION AS A PERCENTAGE OF GRAVITY.

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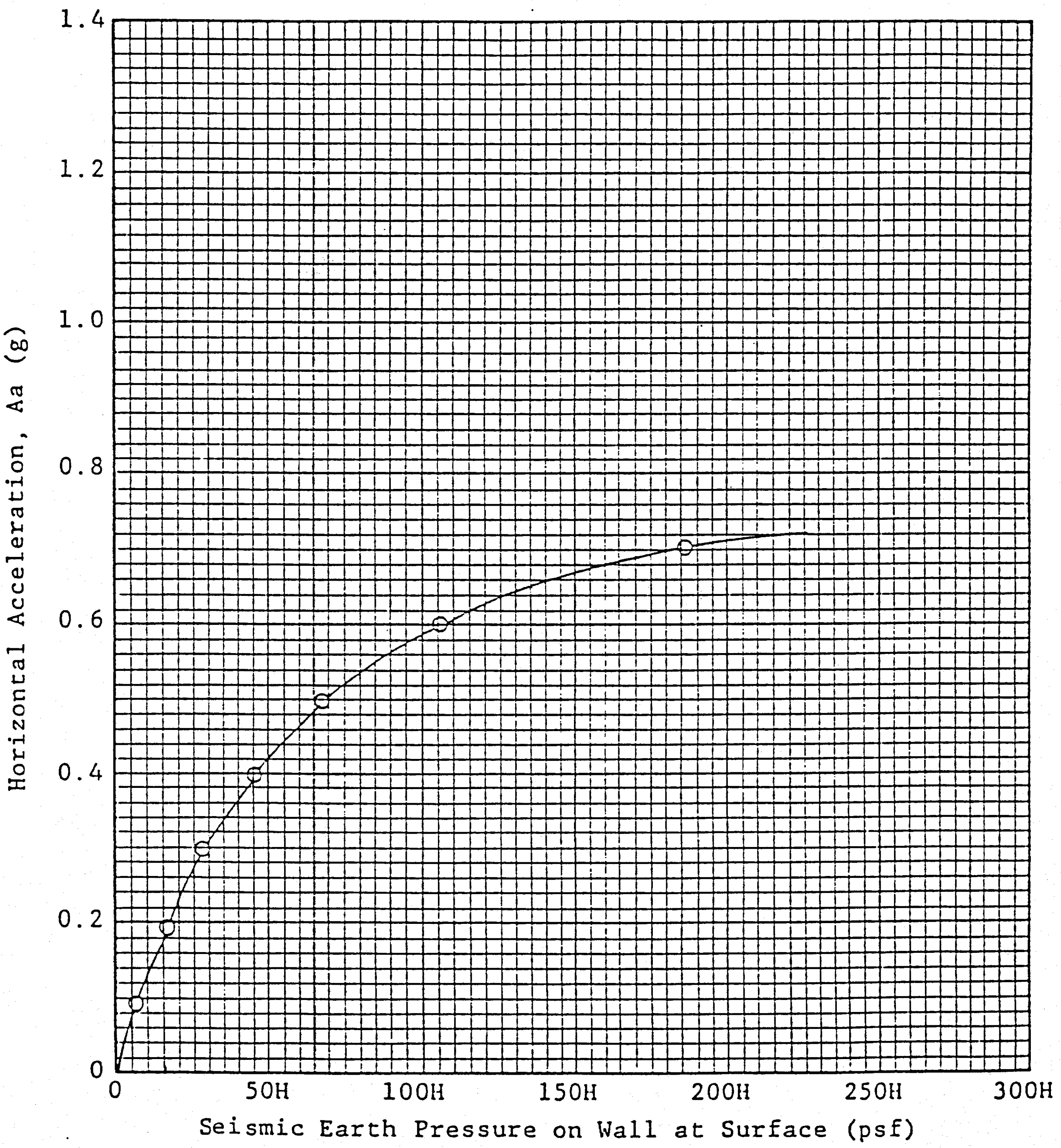
FIGURE 16

SEISMIC EARTH PRESSURE AT SURFACE
VERSUS HORIZONTAL ACCELERATION (Aa)



gare presented, based on a return period of 500 years. The relationship between Aa and earth pressure at the surface is presented in Figure 16. This allows selection of a design recurrence interval, obtaining the corresponding value of Aa from Figure 13 and constructing seismic earth pressure diagrams for design. Walls should be designed with a minimum factor of safety of 1.5.

Recommendations apply to walls with horizontal backfill. Recommendations for sloping backfill will be given upon request.

Backfill behind walls should consist of free draining, well graded sand and gravel with nonplastic fines, meeting the requirements given in Appendix D. Free drainage should be provided with the use of drain pipes at the base of the wall. A geotextile filter (Typar 3401, Mirafi 140N, Trevira S1127) or an approved equivalent should be placed between the granular drain and the soil.

6.7 Site Drainage & Moisture Protection

Although none of the soils along the conveyor line corridor were found to be severely moisture sensitive (with the exception of expansive clay pockets near the surface which will be removed), postconstruction moisture increases could cause slight additional settlements. Thus, positive site drainage should be provided during construction and maintained thereafter. Runoff should be carried away fom the structures by nonerosive devices at the ground surface so erosion and ponding of water does not occur. In no case should long-term ponding of water be allowed.

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TABLE 2

Summary of Geotechnical Conditions & Recommendations

Conveyor No.	Approximate Station	Maximum Cut/Fill	Description of Geotechnical Profile	Recommended Cut/Fill Slopes	Estimated Excavation Methods
6	Sta.163+82 to Sta.173+52	Conveyor on Truss	Man-made Fill	-	-
7	Sta.0+00 to Sta.6+10	40' fill	Drainage in Harkers Alluvium	2:1 Fill	-
7	Sta.6+10 to Sta.7+50	Conveyor on Truss	Predominantly Harkers Alluvium	2:1 Fill	-
7	Sta.7+50 to Sta.12+20	40' fill	Gentle to Steep Slope on Harkers Alluvium	2:1 Fill	-
7	Sta.12+20 to Sta.14+50	10' cut	Gentle to Steep Slope on Harkers Alluvium	1.5:1 Cut	Scrapers, light ripping
8	Sta.4+10 to Sta.6+70	10' fill	Small Drainage in Harkers Alluvium	1.5:1 Fill	-

Note: Missing station intervals are at or very near existing guide.

TABLE 2 (CONT'D.)

Summary of Geotechnical Conditions & Recommendations

Conveyor No.	Approximate Station	Maximum Cut/Fill	Description of Geotechnical Profile	Recommended Cut/Fill Slopes	Estimated Excavation Methods
8	Sta.6+70 to Sta.11+20	50' cut	Moderate Slope on Harkers Alluvium	1.5:1 Cut	Scrapers, light ripping
8	Sta.11+20 to Sta.12+90	15' fill	Small Drainage in Harkers Alluvium	2:1 Fill	-
8	Sta.12+90 to Sta.17+50	35' cut	Shallow Harkers Alluvium and Exposed Bedrock	1.5:1 Cut	Scrapers, light to heavy ripping or possible blasting at depth in localized zones
8	Sta.17+50 to Sta.20+55	25' fill	Drainage in Harkers Alluvium overlying bedrock	2:1 Fill	-
8	Sta.26+00 to Sta.42+10	70' cut	Shallow Harkers Alluvium overlying bedrock	1.5:1 Cut	Scrapers, light to heavy ripping or possible blasting at depth in localized zones
8	Sta.45+00 to Sta.46+45	15' fill	Moderate Slope on Harkers Alluvium	1.5:1 Fill	-

Note: Missing station intervals are at or very near existing guide.

TABLE 2 (CONT'D.)

Summary of Geotechnical Conditions & Recommendations

Conveyor No.	Approximate Station	Maximum Cut/Fill	Description of Geotechnical Profile	Recommended Cut/Fill Slopes	Estimated Excavation Methods
8	Sta.45+00 to Sta.46+45	15' fill	Moderate Slope on Harkers Alluvium	1.5:1 Fill	-
8	Sta.46+45 to Sta.50+60	35' cut	Moderate Slope, Shallow Harkers Alluvium and exposed bedrock	1.5:1 Cut	Scrapers, light to heavy ripping or possible blasting at depth in localized zones
8	Sta.50+60 to Sta.54+30	65' fill	Canyon in Harkers Alluvium (Barneys Wash)	2:1 Fill	-

Note: Missing station intervals are at or very near existing guide.

7. DISCUSSION & RECOMMENDATIONS FOR EARTHWORK

For purposes of discussion and presentation of recommendations, the conveyor line corridor has been divided into sections on the basis of simular geotechnical conditions and required cut and fill as indicated on Sheet 2. These sections are presented in Table 2.

From the exit portal of the existing 5490 railroad tunnel to the first transfer station (sta. 173+52) the conveyor is underlain by a varying thickness of manmade fill.

The remaining portion of the alignment is underlain by Harkers Alluvium of varying thickness underlain by bedrock. Bedrock may be present at shallow depth or at the surface in several locations along the alignment.
Where the corridor crosses the Harkers Alluvium, excavations and cuts can be made with little difficulty using conventional earth moving equipment. In areas where bedrock is exposed or shallow, excavations can probably be made with light to heavy ripping. Blasting may be required in some of the deeper cuts along the alignment.

All of the material which will be excavated appear to be suitable for use in embankment fills. Some selective blending may however, be required to provide Class I structural fill.

No geologic faults which are active in the engineering sense were encountered along the proposed alignment.

7.1 Cut Slopes & Embankments

Table 2 presents recommendations for cut slopes into existing soils and rock as well as recommended slopes for embankment fills.

7.1.1 Slope Erosion and Cut Ditches

A considerable amount of erosion is expected on most cut and fill slopes. Drainage ditches should be provided at the top of cut slopes to minimize surface runoff down in the slopes and subsequent erosion. Except where rock-fill is involved, the use of drainage berms and lined drainage channels should be considered for the design of embankments.

Cut ditches at least 10 feet in width are recommended to allow cleaning of eroded material without interference with service roads. A non-scouring velocity of 2.5 feet per second is recommended for the design of cut ditches in the Harkers Alluvium.

7.1.2 Slope Benches

It is recommended that slope benches at least 12 feet wide be considered for cut slopes deeper than about 50 feet. The benches should be at one-half the slope height or 50-foot intervals, whichever is less. The purpose of the benches would be to control slope erosion and rock-falls, improve drainage and allow access for slope maintenance.

7.1.3 Geotechnical Conditions for Excavations

To aid in cost estimating, estimates are presented in Table 2 concerning the methods necessary to make excavations for each of the cut sections. The estimates were made on the basis of exploratory boring information, compression wave velocities from refraction seismic work, examination of surface exposures of geologic materials and deep exploration of the Harkers Alluvium at the concentrator site.

7.1.4 Grading Requirements

All excavated materials are suitable for use in fills with the exception of topsoil. Different compaction criteria are recommended for soils and rock-fill (see Sections 7.1.7 and 7.1.8).

7.1.5 Clearing, Stripping and Surface Compaction

Where fills are to be placed, the existing ground surface should be cleared of vegetation and the surficial organic topsoils stripped. Excavation of surface soils should include all soft soils, moisture sensitive materials, soils weakened or disturbed by grading or traffic of construction vehicles, and all soils containing roots, vegetation or significant amounts of organic material.

The upper 6 inches of exposed native soils in areas to receive additional fill should be scarified, brought to within 2 percent of optimum moisture content, and compacted to 90 percent of maximum dry density as determined in accordance with ASTM D1557.

Where clean cut surfaces on undisturbed rock or very strongly cemented soils are present, the surface need not be scarified and compacted.

7.1.6 Special Treatment of Steep Embankment Foundations

Where embankments must be constructed on natural slopes steeper than 5:1 (horizontal to vertical), the embankment foundation soils should be benched or terraced in a manner so that a firm bond is achieved with the fill being placed. Benches should be no less than 2 feet in vertical height nor more than 4 feet.

7.1.7 Compaction of Fine Grained Soils

Where predominantly fine grained soils are utilized in embankments, the fill should be compacted to a density equal to or greater than 90 percent of maximum dry density as determined by ASTM D1557. Moisture content during compaction should be maintained within the limits of 2 percent below to 3 percent above optimum moisture content.

Compaction should be performed utilizing lifts no thicker than 8 inches unless the contractor can demonstrate that the proper degree of compaction is being achieved with thicker lifts.

7.1.8 Compaction of Rock-Fill

Rock-fill which may be obtained from cuts can be utilized as fill as follows.

The material placed in the main part of the fills should be well graded and contain no particle sizes larger than 2.5 feet in least dimension, except on the slope faces where larger sizes may be placed as riprap. The material should be free of vegetation, organic matter, debris and other deleterious material.

Required compaction should consist of a minimum of four passes with a vibratory-type tractor pulled roller having a minimum weight of 10 tons. At least 90 percent of the weight should be transmitted to the ground through a single steel wheel. The drum should have a diameter between 60 and 66 inches and a width between 72 and 80 inches; and the weight of the vibrating portion (including the drum, shaft and internal machinery) may not be less than 12,000 pounds. The frequency of vibration during operation should be between 1,100 and 1,500 cycles per minute (cpm), and the dynamic force at the operating frequency should not be less than 40,000 pounds. The roller should be towed at speeds not to exceed 2.5 miles per hour.

Compacted thickness of each lift should be no more than 3.0 feet. In lieu of the roller-type and number of passes specified above, alternative compaction equipment and procedures which will impose equivalent compactive effort, approved by the geotechnical engineer, may be employed.

All rock-fill should be thoroughly and uniformly wetted prior to compaction such that, during compaction, the moisture content of the fraction of rock-fill passing the 3/4-inch sieve will be maintained at or above the optimum moisture content as determined in accordance with ASTM D1557.

7.1.9 Prospective Borrow Sources

It appears that all material derived from recent alluvium in main drainage deposits of Harkers Alluvium will be acceptable for use as fill, provided the material is free of debris and vegetation. Some selective blending may however, be necessary. Thus, borrow can be obtained by cut widening in these soils or from borrow pits adjacent to the alignment.

7.1.10 Granular Roadway Surfacing

It is recommended that all service roads be covered with a layer of crushed rock or aggregate base course at least 12 inches thick.

The surfacing material used should meet the following grading requirements as determined by ASTM C136.

Sieve Size (Square Openings)	Percent Passing by Weight
1 1/8 inch	100
1/2 inch	79-81
No. 4	49-61
No. 16	27-35
No. 200	7-11

The plasticity index should be nonplastic.

Unless strongly cemented soils or rock are present, prior to placement of the surface course, the upper 6 inches of the roadway subgrade should be scarified and compacted to a density of at least 90 percent of maximum dry density as determined by ASTM D1557.

8. RECOMMENDATIONS FOR CONCRETE

The results of chemical testing on selected samples obtained during the investigation are presented in Appendix B. The results indicate the existing fills located near Bingham Canyon (Borings C1, C2 and C3) may have been contaminated by leachate which have resulted in low pH values and high percentage of total water soluble sulfates. Considering these results, cement utilized in this area should be Type V. The remaining areas are underlain by Harkers Alluvium and bedrock. The results from these areas indicate negligible sulfate attack on buried concrete.

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